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PAPERS, REPORTS, DISCUSSIONS, AND MEMOIRS

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ECONOMIC CHANGES IN INDUSTRIAL LIFE

By Martin J. Insull,* Esq.

According to an eminent professor of that day the world of 1822 was considered a complex one. This Mr. Insull attributed to the manufacture of the steam engine as developed and improved by James Watt, in 1774. In 1807, Fulton's steamboat had made its trial trip up the Hudson River and in 1824 "the waters of the United States were thrown open to the free navigation of steamboats by a decree of the United States Supreme Court". In 1825, George Stephenson opened the first steam railroad line, and the first steamship crossed the Atlantic Ocean in 1830.

Summarizing this historic picture, Mr. Insull remarked that,

"These developments in the use of steam began the revolution in the industrial life of the world which has been continuous since the invention of James Watt and which has resulted in problems, both political and sociological, that have yet to be solved. Previous to this revolution, the manufacturing of the world was done by the individual craftsman serving a small clientele located near the workshop. He has long since passed, and with him have gone the contemporary sailing vessel and stage coach. In their places we find great manufacturing plants, fast steam trains, and ocean liners, none of which, as we know them to-day, was any part of the world of 1822."

Centralization as a Factor.—Thus, the development of power in its various forms caused, he believed, "the centralization of industry and the movement from the land to the cities." He then outlined the early history since 1822 and the present economic opportunities of the State of Texas. With its smelters, factories, mills, gins, oil wells, pipe lines, refineries, mines, farms, and ranches, "Texas, surely, presents striking differences from the world of 1822."

Expanding his theme further, Mr. Insull then stated:

"Likewise, throughout our nation, complexities of that day are trifling simplicities compared with the complexities of to-day. Since then we have had the telegraph, telephone, the development of manufactured gas and the discovery of natural gas, electric light and power, the electric railway, the automobile, the radio—and now, the airplane. * * * We have harnessed our water powers, tapped our supplies of oil, increased the efficiency of our mining and use of coal, and developed mass production of manufactured products; and we are now rapidly going into massed merchandising."

Human Endurance as a Factor.—Basically, the entire modern industrial organization is being impelled by one machine, the human being. Is he

* Pres., Middle West Utilities Co., Chicago, Ill.

standing the strain or is the human machine breaking down? The answer can be only hypothetical since apparently there is no choice but to continue. In a casual scanning of the obituary columns of a New York newspaper, it seemed that nearly all the men died of heart failure. This is an economic loss to be reckoned with since it indicates that the strain of modern life is removing some men, "whose vision, experience, and mature judgment we may need really more than we appreciate".

The Steam Turbine as a Factor.—There are advantages as well as disadvantages, however, and (quoting Mr. Insull):

"Some of the complexities we have acquired in the last century are overcoming some of the disadvantages of that period. While the tendency of the early days of steam development was toward centralization, it has brought its own cure for this. In the last twenty years has come the marvelous development of the steam turbine, which has done as much as anything else to make possible the general distribution of electrical energy all over the country, even to the smallest hamlet and farm."

This has made possible the delivery of power practically everywhere and it is no longer necessary to centralize as in the days when ease of fuel transportation was of greatest importance. The finished product can now be manufactured at the source of the raw material and the same power that caused centralization is now active in correcting its evils. However,

"With power and transportation both available at any point, the excessive movement of power increases, and since power is the cheapest element in industry to move, this opens the road to final efficiency in manufacture and distribution."

Corporate Ownership and Federal Legislation as a Factor.—Each development since the time of Watt has involved larger and larger units of organized business until modern industry has left the field of individual ownership and entered that of factory expansion, steamboat development, and railroad building; and after the Civil War this country experienced a great general industrial expansion similar to that following the World War.

At first, these industrial corporations were inclined to retain the autocratic attitude of the early factory system until the public rebelled and created laws to correct this condition. Quoting again:

"The business legislation which resulted, however, was not founded on a thorough study of the conditions to be corrected * * * and there has been, as a result, a considerable reversal of public opinion. * * * In that period we have seen beneficent changes in the ethical standards of business. * * * The outcome of these changes has been public appreciation of the principle that capital invested in business—which, as a matter of fact, belongs to the public—is entitled to fair wages. * * * This has brought about a far better understanding between large business enterprises, their employees, and the public."

Confidence as a Factor.—Mr. Insull stressed the importance of preserving this confidence by a frank revelation of facts:

"The most successful businesses are those that are the most informative", he declared. "If our machine civilization does not produce the greatest good for the greatest number then we are on the wrong track. * * * If mass production in the ultimate circumstance means that everything will be done by

machinery, and practically nothing by human labor, then we destroy the very consumer purchasing power that makes mass production possible. There may be some need of a regulatory influence to avoid these smashes that come from going too fast on unknown roads. * * * Sincere discussion of these matters between business, labor, and the consuming public * * * will establish a confidence that we are all working for the general good."

Labor as a Factor.—Unemployment is on the increase, he declared, but the industrial development of aviation will absorb much of the excess labor released from other industries as a result of greater machine efficiency. Mr. Insull then raised the question as to whether new demands for labor would be created indefinitely as the mechanical efficiency of each industry continued to increase and the need for labor decreased in proportion. These problems will be solved by frank recognition and publicity. Mr. B. C. Forbes, the financial analyst, recently predicted that America was rapidly becoming a nation of employees. The situation promises to become seriously complicated if all these employees should become dissatisfied with the treatment received; or as Mr. Insull put it:

"If the economic development of business requires the disappearance of the manager-owner, something must be done to make good his loss by giving him opportunities to become an owner in the business that employs him."

In closing, he stated that:

"This great country has been elevated to its pre-eminent position by personal initiative, operation, and ownership. That is the heritage handed down to business to keep unsullied, either by its own acts or by the acts of others. It cannot be so kept unless business men conduct themselves and their businesses for the public and National good. The responsibilities of the industrial army for the National good in times of peace are, if anything, greater than those of the military army in times of war. Any mistakes in strategy and tactics in the industrial army may bring greater and more far-reaching defeats to the Nation than any of its military army could possibly suffer. 'He who refuses what is just, gives up everything to him who is armed.'"

MODERN METHODS OF DEEP WELL-DRILLING

By W. W. Scott,* Esq.

After describing briefly the historical development of well-drilling methods in the Southwest, Mr. Scott gave a résumé of present practice. He defined the term, "Southwest," as the region including the States of Arkansas, Kansas, Louisiana, Oklahoma, and Texas. This region now produces about 60% of all the oil pumped daily in the United States. Drilling methods used are the same as those in all parts of the country. Most of the changes in methods and equipment have been developed to meet the need of drilling deeper wells.

Not long ago an oil well drilled to 4 500 ft. was considered a deep one, but recent developments are such that holes drilled to 6 000 ft. are a common occurrence. The deepest producing well in the world is now at Big Lake, in West Texas, 8 520 ft. There are two methods of drilling wells, and these are commonly referred to as the cable tool method and the rotary method. Mr. Scott

* Petroleum Engr., Humble Oil & Refining Co., Houston, Tex.

states that "when discussing modern methods of drilling, one is compelled to consider principally the rotary method."

In regard to derricks and other equipment, Mr. Scott stated that,

"Within the past few years there has been a complete transition from the use of wooden to steel derricks. Practically all derricks for use in rotary drilling are 122 ft. high and are made either of tubular or angle iron. The design is along lines of good engineering practice and present derricks are of sufficient strength to withstand the loads necessary to drill to depths of 8 000 or 9 000 ft. The following tabulation will give an idea of the loads handled while drilling a well of ordinary depth:

Kind of pipe.	Size, in inches.	Weight per foot, in pounds.	Length, in feet.	Total weight, in pounds.
Drill pipe.....	6⅝	22.2	4 000	88 800
Drill pipe.....	5 9/16	19.0	5 000	95 000
Drill pipe.....	4½	16.6	5 000	83 000
Casing	8⅝	36.0	4 000	144 000
Casing	7	26.0	5 000	130 000

"Modern derricks for deep well work are built to withstand loads as high as 1 000 000 lb."

Casing and Drill Pipes.—One important factor in the development of deep-well drilling methods, according to Mr. Scott, has been the introduction of seamless steel casing and drill pipe. One type is a heat-treated drill pipe, and another is a special alloy pipe intended for use in casings drilled to especially great depths. The walls are designed thick enough to withstand the collapsing pressure to be encountered, and the joints are made strong enough to hold the string together. The characteristic of one of these alloy pipes for the more common sizes is, as follows:

Size of casing, in inches.	Weight per foot, in pounds.	Length of string equivalent to collapsing strength of casing, in feet.*	Length of string equivalent to tensile strength of joint, in feet.*
7	26	3 418	6 226
8⅝	36	2 908	6 340
10¾	45	2 030	5 919

* Safety factor of 2½.

Changes in Drilling Equipment.—One improvement in equipment has been the introduction of an automatic control device that maintains a positive torque in drill pipe. This is accomplished by controlling automatically the weight or pressure on the drill bit. It eliminates the dependence on human elements, and as a result the holes are straighter and there is less wear on drill pipe and casing. In addition to the introduction of new equipment and devices, Mr. Scott mentioned many refinements during the past five years. For instance, almost all equipment was formerly made of cast iron, and the moving parts were equipped with common Babbitt bearings. To-day, drilling equipment is made of cast steel, with roller bearings at movable parts of the same quality as those used in the best makes of automobiles.

Of motive power, Mr. Scott declared:

"Improvements have been made in oil-field boilers. During the past ten years, the size has been increased from 45 h.p. to as large as 125 h.p. for an individual unit. By necessity, oil-field boilers for drilling are portable and are moved from place to place at short intervals of time. The old type of 45-h.p. boiler weighs 10 000 lb., the 100-h.p. weighs 26 000 lb., and the 125-h.p. weighs 27 500 lb. Each drilling rig requires from 150 to 300 boiler h.p. The 75 and 90-h.p. boilers are most popular because larger sizes are unhandy to transport.

"Five years ago steam pressures of 100 lb. were used in oil-well drilling, while now the pressure requirement is 250 lb. and, in some cases, as high as 300 lb. This change has come about as a result of drilling to greater depths and due to keen competition in drilling, which necessitates the saving of time. The use of high steam pressure has resulted in a change in the design of steam valves for engines and pumps. The tendency is to use the piston type of steam valve.

"Drilling engines capable of developing 300 h.p. are used on all rotary drilling rigs of the modern type. Slush pumps have also undergone changes and improvements. Ten years ago, slush pumps were made to handle maximum pressures of 500 lb. per sq. in., but now they are made to handle pressures of 1 500 lb."

In regard to power selection, Mr. Scott stated that,

"The proper selection of power to be used on any drilling well is still a question of debate. Steam is being used on the majority of the wells now being drilled. However, from the standpoint of power costs, it is usually the most expensive. Electricity and internal combustion engines are coming more and more into use in drilling operations."

Steam.—The advantage in the use of steam is that most of the workers in oil fields are familiar with it. Its use is entirely dependent on available water supply. The lack of this supply makes necessary the use of gas engines or electric power.

Electricity.—As a source of power, this has been one of the outstanding factors in modern development of deep-well drilling. The cost of electric power necessary to drill a well 3 500 ft. deep is about $625 in the Gulf Coast District.

Gas Engines.—The most extensive use of this type has been on portable outfits intended for shallow drilling and for exploration work. "Gas engines of the automotive type have not been used extensively for drilling with the rotary, but they have been used with some satisfaction." Compared with steam power, costs are lower.

"However, the drilling crews are, as a rule, unfamiliar with the operation of gas engines and, therefore, the care and repair items often become of serious considerations. Because of this, operators will often use more expensive power rather than train a crew to handle gas-engine equipment.

"Diesel Power.—The use of Diesel engines for rotary drilling has been limited. However, it is believed that the Diesel engine will be used in the oil fields in the future, and particularly in areas where steam power is expensive and electricity is not readily available."

Referring to crooked holes, Mr. Scott stated that it has only been very recently that operators have been at all active in determining the extent to which holes depart from the vertical. It has long been known that holes drilled by the rotary method were not straight, but the actual extent of the crookedness has been a surprise to many. Surveys have shown that holes sometimes

vary as much as 40 or 45° from the vertical. Those that vary no more than 10° are considered straight. Mr. Scott contends that:

"Drilling straight holes is a problem to which the executives, practical field men, and engineers are applying their best efforts at the present time. A satisfactory solution has not yet been developed, but much progress has been made."

Methods and Costs of Drilling.—In Kansas, the cable tool method of drilling is still used, and the maximum depth is about 3 200 ft. For a 2 500-ft. well the cost is about $27 000, and for a well 3 200 ft. deep, the cost is about $35 000.

Both the cable tool and the rotary method of drilling are used throughout the State of Oklahoma; however, during the past two years, in an intensive drilling campaign carried on in the Seminole Area, the rotary method was used. The wells that have been drilled in this area are much deeper than past requirements in that and other sections of the State. The rotary method of drilling is now used to a depth of approximately 4 000 ft. and, in every case, the wells are completed with cable tools. The depths of the wells in the Seminole District range between 4 100 to 4 600 ft. The cost will vary from $57 000 to $65 000 per well, or at the approximate rate of $14 per ft., Mr. Scott stated.

Referring to practice in West Texas, he declared that,

"Early drilling in the new West Texas fields was done by the cable tool method; however, in the Hendricks Pool, in Winkler County, the rotary method of drilling replaced cable tools. This was due to the inability of the operators to drill high-pressure gas sands by the cable tool method. The depth of the wells is about 2 900 ft. and the cost is approximately $32 000, which amounts to about $11 per ft.

"In drilling the well in West Texas to a depth of 8 525 ft., cable tools were used throughout. Standard machinery was used and the only special equipment required was extra brakes on the sand reel and bull wheels for extreme depths. The over-all drilling time was 1 030 days, with 870 working days. The cost of the well was $141 500, or at the rate of $16.60 per ft."

* * * * * * * * *

"The cost of drilling throughout the Gulf Coast Area varies a great deal due to the different depths at which oil is found. For instance, wells at a depth of 3 800 ft. will cost about $24 000, while wells about 5 000 ft. deep will cost about $40 000. The maximum cost for wells 5 000 to 6 000 ft. deep will not average more than $10 per ft. and wells of about 3 000-ft. depth will average about $6 per ft.

"In Northern Louisiana and Arkansas the maximum depth is about 3 500 ft. and the cost of drilling is about the same as in the Gulf Coast Area."

It was pointed out that,

"The problem of drilling wells to depths as great as 10 000 ft. is often discussed and, in a number of instances, preparations have been made to drill to these depths. Those intimately acquainted with drilling often speculate relative to the improvements over present drilling equipment that will be necessary to reach that depth:

"1.—It will be necessary to strengthen the derricks.

2.—The power plants will have to be enlarged.

3.—Casing will have to be developed to withstand extreme collapsing pressure and joints for holding long strings of casing together will need to be improved.

4.—Drill pipe for use in rotary drilling will need to be improved.
5.—Methods of cementing will need to be modified.
6.—Improvements will be necessary in all drilling equipment, such as slips which hold the drill pipe, drilling lines, drill bits, high-pressure slush pumps, and design and material in brake flanges in rotary hoists.
7.—Better knowledge of drilling, in order to prevent crooked holes, will require additional information on the action of various bits in different formations, and a better understanding of the cause of crooked holes."

DISCUSSION

DRILLING COSTS

By Walter H. Meier,* M. Am. Soc. C. E.

The costs and weights of the several types of equipment and the comparative cost of drilling wells was the gist of the discussion by Mr. Meier. For those interested in the history of deep well drilling he recommended a paper by Mr. R. B. Woodworth, entitled "The Evolution of Drilling Rigs".†

Future Progress.—Improvements, he stated, had been due mostly to the fact that trained engineers had been introduced into the producing end of the oil industry and that they had brought about a spirit of co-operation between the practical man in the field and the manufacturer of equipment. Even greater improvements will be made within the next few years. Regarding the crooked hole problem, according to Mr. Meier:

"The effort is now being made to better control the direction of the drill hole and keep it vertical. A close study toward the improvement in design of drill bits is likewise being made, with the aim of keeping the bit longer at the bottom of the hole with consequently less time spent in letting it down and hauling it out to change bits. Using present cone or roller rock bits which cost approximately $100 per set, it is not uncommon, in hard rock, to wear out a set in drilling 10 ft. of hole."

Rotary Drills.—In the future, the rotary drilling rig will probably be patterned after the diamond core drilling machine. It will be a self-contained unit, erected on a derrick floor, operated by internal combustion engines or by electric power. New developments may be expected, according to Mr. Meier, in the way of extremely hard and tough metal cutting edges instead of the expensive black diamonds now used. To quote directly:

"Such equipment is already available and has successfully drilled holes to 4 800 ft. in depth against a gas pressure in the hole of 1 700 lb. per sq. in., the hydraulic cylinders on the rotary table having a lifting capacity of 150 tons with 800 lb. per sq. in. water pressure."

For drilling to depths as great as 4 000 to 5 000 ft., the following outfit, costing about $45 000 is generally used:

* Chf. Engr., Atlantic Oil Producing Co., Dallas, Tex.
† *Transactions*, Am. Soc. of Mech. Engrs., 1916.

Three 85 or 90-h.p. locomotive type boilers.
One 11 by 12 twin-cylinder drilling engine.
Two 14 by 7½ by 14 duplex mud pumps.
One 6-in., upset, seamless drill pipe.
One reinforced steel derrick, 122 ft. high, with a 24-ft. base.
One boiler feed-water pump.
Three-speed heavy-duty draw-works.
Water tanks, fuel tanks, derrick lighting equipment, etc.

The comparative costs of prime mover equipment for heavy rotary drilling rigs were given by Mr. Meier, as follows:

Power source.	Weight, in tons.	Cost.
Steam	23	$9 000
Gas engines	8.5	11 000
Oil engines	14	13 000
Electric motors	10.5	7 500

The costs of cable tool drilling, by means of these prime movers, were quoted as follows:

Power source.	Depth of well, in feet.	Drilling time, in days.	Cost of fuel and water.	Unit cost per foot.	Power, in kilowatts.	Fuel, in gallons.
Steam	2 460	59	$3 900	$1.58	.....	.....
Electricity	2 700	41	.....	0.195	23 424	.....
Internal combustion engines	2 978	51	.....	1.39	.....	2 805
Electric power	2 884	41.5	.....	0.215	21 418	.....

Whether compared with steam or internal combustion engines, electricity is the cheapest prime mover and steam is the most expensive. In the judgment of Mr. Meier:

"The objections to internal combustion engines have been the relatively short life and lack of reserve power, such as can be obtained with steam or electric equipment. However, it is quite probable that with the recent development of light-weight, slow-speed Diesel engines, this type will be a close competitor where electricity is not easily and cheaply obtained.

"Operators are beginning to give considerable attention to the power conservation even where free fuel, gas, or oil is available at the wells, and considerable reductions in drilling costs are realized through using all-steel equipment with anti-friction bearings, insulated boilers, and steam lines, installing water-softening equipment, and other improvements.

"Ten years hence, to reach depths of 8 000 and 10 000 ft., at which we will then be drilling, modern methods of deep-well drilling will be as much improved over to-day's, as to-day's methods and equipment are superior to those of ten years ago."

STANDARDIZATION IN WELL-DRILLING PRACTICE

By J. Edgar Pew,* Esq.

The steady and marked increase in the production of oil was pointed out by Mr. Pew in discussing the paper by Mr. Scott. In 1859, the first oil was dis-

* Vice-Pres., Sun Oil Co., Dallas, Tex.

covered and, in 1900, about 68 000 000 bbl. were being produced. It was then thought that all the oil had been discovered. The U. S. Bureau of Mines declared that no oil was to be found in Texas, Arkansas, or Louisiana. In 1901, oil was discovered at the Spindle Top and the estimate of the available supply from this source was a maximum of 200 000 000 bbl. The production of oil in the United States was 400 000 000 bbl. in 1922 and, at present, 700 000 bbl. per day are being produced. If the necessary storage space were available it would probably be possible to produce 3 000 000 or 4 000 000 bbl. per day.

"The present large production of oil", said Mr. Pew, "is largely due to our ability at this time to drill wells at a depth which fifteen and twenty years ago was hardly thought possible. At that time we did not have any pipe that could be used at depths of more than 3 000 or 3 500 ft."

In order to progress in this direction, The American Petroleum Institute has undertaken to bring about an improvement in well-drilling materials. Among the problems to be solved are those concerned with pipes, proper cordage, sufficient power on top of the floor, etc.

Standardization of Pipe.—This factor was the first to be considered because the depth to which wells can be driven depends on the integrity of the pipe. One important need is to obtain standards such that pipes of different manufacturers can be interchangeable. This has been accomplished so that wells can now be driven to 8 000 and even 10 000 feet.

This improvement has been accomplished mostly since about 1924 by the use of better material, such as alloy steel, better joints, seamless casing, etc.

Standardization of Derricks and Other Working Equipment.—Another important phase of this work applies to derricks, rigs, rig irons, boilers, tool joints, etc. There are now standards in use, as follows: One standard tool joint, standard threads on pipes, standards in rotary equipment and rig iron work, such that it is possible to eliminate many parts that were formerly necessary. Altogether, as Mr. Pew stated:

"It has been the effort of the oil industry to study economics, I think, in the last five years, probably more than in any other industry, because we were very backward in doing it prior to that time. Now that we have adopted these standards, we are trying to increase their efficiency by the use of our Petroleum Engineering Division. We hope in this way to put the oil industry on as efficient a basis in its various operations as is any other industry."

Cost of Wells.—In his opinion Mr. Scott under-estimated the costs for oil wells in the Southwest, except perhaps in Texas. The average wells in the Seminole Field which are about 4 000 ft. deep, would cost about $75 000 rather than $50 000. The average deep well in California, about 4 000 or 5 000 ft. deep, costs $100 000, or more.

Re-Opening Old Wells.—Considerable interest was evinced as to the possible salvage value of old oil wells that were sunk before the present deep well-drilling methods were introduced. Mr. Pew called attention to the fact that at Spindle Top, the oil pumped was known as "cap-rock" oil, because of the rock covering a salt dome at the lower depths. Whereas, formerly, these shallow domes could be seen on the surface, many additional ones are being discovered by modern drilling methods.

The result has been that, within 1 000 ft. of the old shallow wells, deep wells have been drilled, thereby permitting the additional production of about 40 000 000 bbl. to date. Of similar conditions elsewhere, Mr. Pew stated:

"At Santa Fé Springs and Signal Hill in California, they thought they had exhausted the big production there which ran between 3 000 and 5 000 ft. Now, they are drilling and have brought those fields back again and are getting production at 6 000 and 7 000 ft. Up in the valley at Capitol Hills, they are getting production approximating 8 000 ft. Nobody thought of drilling that deep five years ago."

In Oklahoma, Mr. Pew declared, there had been wells that were considered exhausted until the advent of modern drilling methods. Then, deeper wells were drilled to new sources of supply; however, the old hole cannot be used for proceeding deeper, because the initial size of casing is not large enough. Except for some recovery of old casing, the old wells have practically no salvage value.

Diameter of Pipe.—The diameter of pipe used depends on the drilling methods, the proposed depth of hole, and the character of formation to be pierced. In Pennsylvania, it was found possible to set a little surface casing and then drill an open hole through rock until the hole reached sand. In the rock the only need for occasional casing was when a large body of water was encountered.

In Oklahoma, a 4 000-ft. hole would require at least five strings of casing, one inside the other. The largest pipe would be at least 20 in. However, if rotary drilling is used instead of cable-drilling methods, it might be possible to use only three or even two strings. A few years ago, well drillers would never have thought of using strings of 13-in. pipe longer than 800 ft. With the present development in alloy steel pipe, strings of 13-in. pipe 4 400 ft. long are being used at Capitol Hills in California. Mr. Pew stated that rotary drilling is fast becoming the accepted method.

Core Samples.—To avoid passing through thin oil strata, it is general practice to take core samples in every possible formation that might bear oil.

GENERAL DISCUSSION

Salvage Values.—Has the development of the 10 000-ft. well, or the 8 500-ft. well, been such as to determine the salvage value of oil fields that were considered drained by the more shallow wells? Is there any salvage value in the old wells? These questions by Fred C. Scobey, M. Am. Soc. C. E., were treated in general by Mr. Pew. However, Mr. Scott replied that the specific answer for any old oil field, would have to be made by a trained geologist. There seems to be good reason to expect deeper "pay" in West Texas and throughout the Gulf Coast District.

As to the salvage value of old wells, Mr. Scott stated:

"In some cases we do use the old well, but I think it is generally good business to start a new well for deeper 'pay'. Usually, the size of the hole in the old well has decreased to a point where it is not possible to go to any

great depth, and the condition of the formation is such that it may not allow you to go ahead without trouble."

As has been noted previously by Mr. Pew, an inquiry by President Marston revealed that development in driving deeper wells has resulted in the renewing of old oil fields in Northern Oklahoma.

Replying to questions by C. E. Grunsky, Past-President, Am. Soc. C. E., Mr. Scott stated that in Southern Oklahoma, where wells are being drilled deeper than 6 500 ft., the practice is to begin with a 15½-in. or a 16-in. casing. At about 4 000 ft., the pipe is reduced to 8 in. in diameter, so that at depths of from 5 000 to 10 000 ft., a 6-in. pipe can be used. It is desirable to have 6-in. pipe as the size at the bottom of the well, because smaller casing makes operating difficult. Many wells, particularly in Oklahoma, are cased with 4½-in. pipe.

Afternoon Session 2:30 to 4:30 P. M.

DEVELOPMENT OF THE OIL AND GAS INDUSTRY IN THE SOUTHWEST

By J. Edgar Pew,* Esq.

Early History.—This paper is a fairly complete history of the discovery, development, and production record of the various fields discovered in Texas, Louisiana, and Arkansas. Mr. Pew stated that the industry had its beginning with a production of 48 bbl. of oil during the entire year 1889, explaining that,

"This was from a shallow well drilled about four miles south of the Town of San Augustine, in San Augustine County, Texas. From 1889 to 1896 the United States Bureau of Mines' *Bulletins* give a total production of 1 811 bbl. of oil for Texas. These same reports give a production of 65 975 bbl. for the year 1896. The increase came from the New Corsicana Field, in Navarro County, Texas."

For the purposes of this paper Mr. Pew divided the territory described into the North Central Texas, East Central Texas, the Gulf Coast District, and West Texas. These he referred to as the Red River Area, Fault Zone, Salt Dome Area, and Salt Basin Area. The districts in Northern Louisiana and Arkansas are referred to by their respective State names. These districts have produced a total of 2 391 755 000 bbl. of oil to January 1, 1929. Of this total, Texas has produced 70%, Louisiana 17%, and Arkansas, 16 per cent. The potential value of oil in Texas was first recognized in 1896, when some shallow wells were drilled in Corsicana. The next development in the Southwest was the discovery of oil in the Spindle Top Field, near Beaumont, Tex. This was the first of the salt dome producing fields, and was the first great development of oil in Texas. It was started in 1901 by Capt. Anthony F. Lucas, in an effort to develop sulfur.

* Vice-Pres., Sun Oil Co., Dallas, Tex.

"Speculation.—The Lucas well flowed wild for a week, at the rate of probably 50 000 bbl. per day, and then quit producing, probably as a result of cavings in the hole. The oil produced caught fire a few days later, and was entirely lost. The well never produced any oil of consequence thereafter. It did, however, 'start something'.

"Captain Lucas thought he held under lease all the area around and on Spindle Top necessary to give him a solid block of leases. It was soon evident that about 70 to 75 acres in the center of his block were unleased. The result was that several of the small tracts composing these 70 acres were to be further divided, and were sold, some of them in as small fractions as one-twenty-eighth of an acre. Hundreds of wells soon started drilling, and as the depth of this production was from 900 to 1 000 ft., there were more wells of a rated capacity of from 25 000 to 50 000 bbl. per day each than had ever been seen in the history of oil."

Speculation ran wild. Stock companies were capitalized in amounts from $100 000 to $5 000 000 with no other holdings than one-sixteenth of an acre, on which was located a well, good for possibly 50 000 bbls. per day. Many millions of barrels sold at 5 cents, and some were sold as low as 1 cent, per bbl. On one single acre at Spindle Top there were 105 wells. While some of these promotions were dishonest, and were made for no other purpose than to obtain money from the sale of stock, the majority of them, thought Mr. Pew, were actuated by faith that they would make money for stockholders.

"Extension of Fields.—Following Spindle Top, new salt dome fields on the Gulf Coast were rapidly developed. Sour Lake, Batson, Saratoga and Humble, in Texas, and Jennings, in Louisiana, all came within the following six years. More of these domes have been developed and others are known and will be developed. Geo-physical instruments, the seismograph and the torsion balance, are used in the discovery of salt domes not otherwise known.

"In 1908 and 1909 the Northern Louisiana fields, near Shreveport, and in 1911 and 1912, the Electra Field, in Wichita County, Texas, were important developments, and because of the higher gravity of their oil, were the first Southwestern oil discoveries of great importance that came into competition with the oil business as it was generally understood by the average man in the oil industry. The Eastern oil men commenced to arrive in Texas in large numbers."

In 1917, a well drilled near Ranger, in Eastland County, began producing 7 000 bbl. of high-grade paraffin oil daily. Later,

"In this county, and in Stephens and Young Counties, there was developed what seemed to be one of the greatest oil fields, in area, and in the size of wells, that had ever been discovered. While these fields produced a large quantity of oil and are still producing many thousands of barrels per day, they probably have made no money, as a whole, for the operators. Wartime conditions, poor labor, high costs of material, and unusually bad road conditions, made the cost of all operations too great for profit."

The next oil-field developments occurred in North Texas, the best of which was the Burkbennet Field, in Wichita County. "Homer and Haynesville, in Northern Louisiana, and El Dorado and Smackover, in Arkansas, were the important developments in these States following that at Shreveport."

Later Discoveries.—In 1920, oil was discovered by the late Col. A. E. Humphreys at Mexia, in East Texas. Subsequently, other parts of this so-called Fault Zone yielded oil, and these fields have already produced nearly

225 000 000 bbl. The Panhandle Area of Texas, in Gray and Carson Counties, was developed in 1924, and has produced a total of 92 000 000 bbl. to January 1, 1929. The Big Lake Field, in Reagan County, was opened in 1923. According to Mr. Pew,

"This may have been the greatest discovery of oil in the United States up to date. The Winkler and Pecos County Fields, and numerous smaller fields in this area, are all important developments arising out of the discovery of the oil possibilities of this Salt Basin Area. The Pecos County Field, better known as the Yates Pool, has a production of about 125 000 bbl. per day, but frequent tests for its potential production, developed but closed in, have resulted in estimates as high as 4 000 000 bbl. of daily production—more than the production of the remainder of the world. Winkler County is only pumping about 175 000 bbl. per day, but it also has a large potential production."

Benefits of Oil Industry.—In referring to the benefits produced by the oil industry, he stated:

"Texas is a vast empire in size, and in every district of any considerable size oil is being produced or the possibilities of oil production are present. As a result of oil in Texas this State is rapidly becoming one of the richest of the Union. It is developing industries on account of cheap fuel and it is building great highways with its millions of dollars collected each year from the gasoline tax. The oil industry in Texas has done more to build up the State than any other industry with the possible exception of the railroads.

"Probably $3 000 000 000 is invested in Texas, Louisiana, and Arkansas, by, and as a result of, the oil industry. This area has more miles of pipe lines and has greater refining capacity than any similar district in the United States."

Natural Gas in Texas.—At first, the natural gas industry did not keep pace with that of oil, despite the fact that they are closely related. Mr. Pew continued:

"While gas has always been found with oil in Texas, and sometimes in large initial quantities, owing to the small area covered by these earlier fields, it had no value, except as a lifting agency for the oil, and its use as a fuel in drilling. Such gas was soon exhausted and did not justify the expenditure necessary to make it available for other commercial uses, even locally. Another reason for the slower development of the natural gas industry in these States was that the earlier known gas was usually from loose sand formations, and it was not thought capable of long, continued production. The finding of gas with better sand conditions has justified the building up of a great natural gas industry, and built on this assured foundation of permanency it has also given a market for the more temporary supplies of gas which, to a large extent, had been dissipated and lost from the producing oil fields."

Natural gas was first used commercially in the Mexia Corsicana District, in 1900. In 1909, gas produced at Henrietta, in Clay County, was piped to Fort Worth, Dallas, and other cities in Northern Texas. The Laredo Field in Southwest Texas was developed in 1911, and in 1911-12 gas was produced at Shreveport, La. Since that time this industry has developed more rapidly so that, in 1929, gas produced from Texas, Louisiana, and Arkansas will be consumed not only in every city and town of importance within these States,

but also in Kansas City and St. Louis, Mo., Denver, Colo., and many other intervening cities and towns.

Gas Production.—As to the production of gas, Mr. Pew pointed out that,

"In 1927, the last year for which the figures have been published, 1 455 428 000 000 cu. ft. of natural gas were produced and delivered to customers in the United States. This was an increase of 10% over 1926, the year of the largest previous consumption.

"The United States exports natural gas as well as oil. During 1927, 184 000 000 cu. ft. of natural gas were exported to Canada and Mexico.

"In 1927, Texas advanced from fourth to second place in the production of natural gas, and, during the same year, showed the largest percentage of increase in production, 45%, of all the major gas-producing States."

The three States mentioned supplied 31 000 000 000 cu. ft. of natural gas to 426 180 domestic consumers in 1927. The income derived was $18 381 000. In addition, there was about 445 000 000 000 cu. ft. sold for industrial purposes, at $37 680 000. In 1927, Texas produced 269 944 000 000 cu. ft. of natural gas.

Gas-Producing Areas.—Mr. Pew stated that,

"The principal natural gas-producing areas in the southwestern part of the United States are the Monroe Field in Louisiana, the Panhandle Field, in Northwest Texas, and the Laredo, or Rio Grande, Field in Southwest Texas.

"To market the natural gas of Texas, Louisiana, and Arkansas, there has been laid and is now under construction a total of 19 740 miles of pipe lines, costing about $300 000 000. These lines are 22 in., 20 in., 18 in., 16 in., 14 in., and smaller, in size.

"In addition, the natural gas industry in 1927 extracted the gasoline from 93% of all natural gas handled. Also, the carbon black industry of Texas, Louisiana, and Arkansas, adds greatly to the figures directly dependent on natural gas. (These investment figures are not available, but are probably between $10 000 000 and $20 000 00.)"

A Great Producing Center.—Declaring that it is evident that the southwestern part of the United States is rapidly becoming the greatest oil and gas-producing area in the United States, Mr. Pew pointed out that it is already the greatest oil-refining center:

"The business of the refineries, and of the oil and gas pipe lines constructed and being constructed, the drilling of wells, the lease rentals, bonuses, royalties, and taxes, both direct and indirect, will total $750 000 000, or more. This sum will be paid in these States by this industry during the year 1929. In addition, the oil-purchasing agencies will distribute about $300 000 000 for oil purchased during the year."

Mr. Pew expressed the opinion that during 1929 no other industry active in this district will contribute directly as much money to its wealth as natural gas.

OIL AND GAS PIPE LINE CONSTRUCTION

By E. C. Kincade,* Esq.

While there is no fundamental difference in designing for both kinds of pipe lines, according to Mr. Kincade, some of the details are quite different.

* Vice-Pres., Gulf Pipe Line Co., Houston, Tex.

For example, joints in a gas line must be tighter to prevent leakage. Therefore, ordinary screw-joints are not sufficient. Gas is pumped by means of compressors while flood pumps are used on oil. Among other differences, Mr. Kincade mentioned:

Different Requirements.—Gas lines need no storage system because the pipe line itself serves for storage, and gas can be produced as consumed; but,

"Oil lines require storage at the initial point of receipt at each pump station and point of delivery. For the benefit of the refineries, it is often necessary to keep different grades of oil separate for ease of refining. Grades of oil are identified by gravity, viscosity, and whether they are sweet or sour. The different grades require separate tankage at each point where they are handled, and often separate pipe lines; but it is common practice to handle more than one grade of oil through a single line."

In theory, the general laws for the flow of gas or liquids are the same, but the elements of surface roughness, dimensions of the line, temperature, specific gravity, viscosity, and topography of the region affect designs differently. For instance, in gas lines water or gasoline is likely to collect in low places, while under similar circumstances oil lines will be affected by the collection of sand or sediment.

Pipe Line Location.—Airplane photography for solving the location problems of a pipe line is being used very widely in Texas, according to Mr. Kincade. For oil lines it is necessary to study differences in elevation in order to predetermine a correct spacing for pumping stations.

"From an economic operating standpoint, pump stations should be located so that the direct operating expense approximately equals the fixed expense on pipe lines. This is governed by the class of pump stations and the character of the oil being handled; for the lighter oils they are about 50 miles apart, and for the heaviest oils, about 15 miles apart."

Welding vs. Screwed Joints.—Comparing the merits of welding and screwed joints, it was stated that,

"In the past, a majority of oil pipe lines have been connected with screwed joints; gas lines have been joined by packed couplings. The present trend is to weld lines and I doubt if there will be another screwed oil trunk line or packed coupling gas line constructed of any considerable length. The problem of securing competent welders has passed, and automatic electric welding for pipe lines is being developed which will further simplify welding. Lines are either electric, or acetylene, welded. Both have about an equal number of sponsors. In welding lines, provision has to be made for expansion and construction, due to temperature changes."

The average temperature encountered is 50° Fahr., and the normal range is from 40 to 80°, which is such as to require a consideration of a possible expansion of 20 in. per mile. Mr. Kincade related how, in the past, this expansion has been provided for on large gas lines, by the packed couplings; and with screw-joints, expansion has been guarded against by laying parts of the line somewhat crooked to leave a certain amount of "slack". The actual amount has been usually a matter of guesswork on the part of the men in the field, and sometimes through faults of construction, this guess has not been accurate enough and the line has pulled apart. He predicts that, in the future, pipe lines, 10 in. in diameter and more, will have expansion joints.

At present, the best type of joint is a corrugated section, welded into the line at predetermined intervals.

Pumping Stations.—Pumps for handling oil do not differ materially from those used in handling water under a pressure of 700 or 800 lb., except that perhaps more precaution is taken to prevent fire. In gas pumping stations the gas is received through filters which take out the sand and scale. The pumps are ordinarily gas engine-driven compressors. The sizes of the pumping stations range from 25 h. p. in small field stations to 300 h. p. on trunk-line stations. About 70% of these are driven by Diesel engines, and the remainder by electricity, gas, or steam.

In conclusion, Mr. Kincade declared that:

"Regardless of power, source, or size of stations, pipe line companies are as careful as to the cost of power as any industry. There is 45 000 miles of oil trunk lines, 6 in. and up, in use to-day, delivering approximately 2 000 000 bbl. of oil daily to refineries, and ships. These lines handle this amount on a weighted average of ten times before delivery. Another 46 000 miles of line, 6 in. and less, are gathering approximately 2 500 000 bbl. from producers and delivering it to trunk lines, railroad tank cars, and refineries. Oil is not only measured from station to station but, in storage, is measured daily."

DISCUSSION

WELDED JOINTS *vs.* SCREWED JOINTS

By F. Ray McGrew,* Esq.

A general discussion of both papers was opened by the comments of Mr. McGrew on the relative merits of welded and screwed joints in oil pipe lines. While he agreed with Mr. Kincade that the present trend is toward the welding of pipe line, he did not agree that all major trunk lines in the future would be welded. Screwed joint construction offers so many advantages over the other that it will probably be used a great number of years. For gas lines, welding is the proper method.

Importance of Ditching.—Quoting Mr. McGrew:

"In the early part of Mr. Kincade's paper, he makes a statement that backs up my idea that screwed joint construction is good. He says 'while they hold crude oil without the slightest signs of leakage when properly made up.' I have found that to be true. I would add to that phrase, however: 'When properly made up and properly ditched'.

"In my estimation in the construction of a pipe line—whether it be screwed or welded construction—the most important phase is the ditching. If a line is properly ditched, all stresses and strains are relieved and you will have little, if any, trouble from pulling apart or faulty construction."

Locating Stations.—He considered that Mr. Kincade's estimate of 50 miles as the best average station distance for the lighter oils is rather high for average topography. While 50 miles may be adequate for a simple line, over

* Pres., Standard Pipe Line Co., Inc., Shreveport, La.

ideal plain, flat country, without any change in elevation, Mr. McGrew declared that 38 miles should be the maximum to be considered. Among the disadvantages of the longer line is the relative cost of increasing the capacity.

River Crossings.—Each problem of crossing a river with a pipe line must be solved as a separate case. However, probably the same material would be used for all conditions. To quote Mr. McGrew:

"It is the usual practice to use double weight pipe. * * * Before the advent of welding, of course, all waterway construction was screw construction. Those joints are protected by heavy cast-iron * * * 'river clamps'. They consist of two saddles of heavy cast iron recessed to receive the collar. They extend out on either side of the collar a liberal distance in order to stiffen the joint and take the stress off the thread. These clamps are held to the pipe by close heavy bolting on either side. Clamps not only give the joint rigidity, but also act as additional weight or anchor for the line. They hold it to the bottom."

In country where the river bed is rock, it is sometimes necessary to use anchor hooks, but when the bottom is mostly silt, the pipe is anchored automatically by sinking it into the mud and soil. In that case, the most important consideration is to provide for the slack on the banks of the river. Mr. McGrew pointed out that:

"Pipe lines crossing the Lower Mississippi and those rivers in Southern Louisiana continually feed into the river. In other words, some lines in our service have been in place for twenty years and they are still sinking in. It is assumed that they never will reach the bottom. So it is necessary, in a condition of that kind, to approach the river and give it a snake-like profile in order that the slack may travel into the river. The heavy weight and the clamps, etc., of the line keep settling to the bottom and if you do not provide slack in liberal quantities the line becomes tight and will pull apart, not necessarily in the river, but on the river bank."

WELDED JOINTS FOR GAS LINES

By F. L. Chase,* M. Am. Soc. C. E.

In discussing Mr. Kincade's paper, Mr. Chase agreed as to the superiority of welded pipes, stating that:

"Within the last three years, the Lone Star Gas Company has laid more than 1 500 miles of gas lines, varying in size from a maximum of 20 in. in diameter. All that line is welded. We have laid between 2 500 and 3 000 miles in distribution plants in about 100 to 125 towns. All that is welded pipe."

During three years of operation, twelve cases of trouble have occurred in that gas line of about 4 000 miles.

Bridges for River Crossings.—Mr. Chase advocated the installation of pipe-line bridges for river crossings, offering the following reasons:

1.—They eliminate trouble due to high water.
2.—There are no pipes pulling apart and repairs to make.

* Vice-Pres., Lone Star Gas Co., Dallas, Tex.

3.—In case of accident, there is no discontinuance due to inaccessibility of the line.
4.—Usually, they are not more expensive than river-bed crossings.
5.—The maintenance costs, except for painting, etc., are practically nothing.

GENERAL DISCUSSION

Replying to Robert H. Merrill, M. Am. Soc. C. E., Mr. Kincade declared that it is proper to rifle the pipe lines if the oil can be driven fast enough to throw the water to the outside. It is not practical in ordinary service, however.

Answering a question by J. H. Brillhart, M. Am. Soc. C. E., as to the comparison of first costs between welded and screwed joints in the same size, Mr. Kincade stated that: "There is very little difference in the first cost of the two lines; it is in favor of the welded construction, if anything."

Supplementing the discussion, W. P. Stine, Assoc. M. Am. Soc. C. E., declared that there are many cases in which the overhead pipe crossing cannot be built. This is true at a great many navigable streams. The type of stream crossing will depend on the circumstances for any particular case.

Mr. Kincade replied to a question by W. E. Hand, Jun. Am. Soc. C. E., concerning steps necessary to prevent corrosion of the metal by oil with a high sulfur content, by stating that tanks in service for two years collected a hydrogen-sulfide scale probably 1½ in. thick. Explanation of the steps necessary for the prevention of its occurrence were considered beyond the scope of the paper.

HIGHWAY AND CONSTRUCTION DIVISIONS

JOINT MEETING

APRIL 25, 1929, 9:30 A. M. TO 12:30 P. M.

HIGHWAY LOCATION PROBLEMS

By Gibb Gilchrist,* M. Am. Soc. C. E.

"When the highway location problem is solved, the highway problem itself is well on its way to solution." Thus Mr. Gilchrist introduced his subject and proceeded to describe the various problems encountered.

Average Life of Highways.—That the life of a highway is dependent, not so much on the type of surface as on its location and the width of its right of way is one of his principles. In other words, the average life should be considered as that period during which a highway can be used as an artery of traffic. Public opinion will eventually demand that such a highway be located with a fair degree of economy.

The Line of Least Resistance.—In the past, highway engineers have been inclined to locate a road parallel to a railroad or interurban traction line right of way, or along section or property lines, without much consideration for the natural terrain or the economy of the route. Some of the blame for this must be charged against mass meetings and conferences of citizens at which the orders for road routings are often framed. Mr. Gilchrist did not offer this explanation as a criticism of road location of that kind, because it indicated a live interest among taxpayers which is very important to the success of any public project. There are, according to Mr. Gilchrist:

"Few cases where the location or construction of any type of road or highway has not at least paid for its investment over the period of years which it has been used. How much greater return, however, could have been obtained from the use of highways if we could have saved, as time went on, the cost of traveling much extra distance; and with how much more safety the public could have reached its goal!"

That almost every important railway line is rather closely paralleled by a State highway is explained by the fact that the railroads have developed the population centers that created the demand for highways. Highway engineers, however, have been too ready to accept a railroad location when a shorter route, even with somewhat sharper grades, would have been more economical.

* State Highway Engr., Austin, Tex.

Aerial Photography.—The Texas State Highway Department uses aerial mapping methods in its reconnaissance surveys. To quote Mr. Gilchrist,

"The usefulness of an aerial survey * * * begins at the first inception of the work at hand and continues until the last shovelful of dirt is in place. It places in the hands of the engineer many of the details that are to be found on the ground and, in addition, gives a broad view of the field of operation that has never been available in any other way. * * * He may make locations directly on aerial mosaics; he may study minutely the locations of drainage structures and other proposed improvements. He may calculate stream flow and plat drainage areas without the use of field operations; and throughout the progress of his work the aerial survey stands by as a constant check upon the more exact field calculations."

Aerial maps are especially useful in planning stream crossings and in determining routes through cities and towns. Single-flight maps on a scale of 700 ft. to the inch cover a strip of terrain about 6 000 ft. wide, whereas double-flight maps will be 9 000 to 10 000 ft. wide. The cost is from $14.00 to $17.50 per lin. mile in the former case and approximately twice these amounts in the latter. After the line has been given a temporary location by walking over a proposed route and by making small changes in a tangent or curve to fit some special local condition, the final location is staked out and the engineer then prepares his alignment, or right-of-way, maps. These show the name and abstract reference to each tract of land. They are usually drawn to a scale of 400 ft. to the inch.

General Locating Considerations.—The important factor in highway location is to select the most direct line between two objectives. It should provide the least number of waterway crossings and the easiest grades and curves possible. On sharp changes in grade, the proper distance between the eye of the driver and the top of an approaching automobile should be not less than 500 ft. In general:

"Such locations should be made and grades prepared with the primary object in mind so that regardless of the development of the country through which the highway may pass any surfacing placed on the prepared grade would only be a stage of improvement, the ultimate life of which would depend upon traffic requirements of the highway. * * * In other words, a highway properly located with a grade line of sufficient flatness to meet the exacting demands of increased traffic, and with curves in easy stages, will have its ultimate life more permanently assured as years go by, and its condition should be better at the end of fifty years than at the end of five. With these considerations in mind the locating engineer, in considering his route between objectives, should disregard 'the line of least resistance' unless he is prepared to see the day come within his own life time—or better still, within a decade—when the route he lays out will be relegated to the status of a lateral or community road, or be entirely abandoned. When such time comes it very likely will be forgotten that the original location was perhaps determined by considerations purely local, and the necessity for relocation or abandonment, may, without fear of contradiction, be explained as the correction of a mistake in engineering."

Location with Respect to Safety, Economy, and Utility of Operation.—This is the most important problem of highway location. Mr. Gilchrist declared first, his belief: "In the democratic principle that the voice of the majority should rule." Never had he known it to fail that:

"Where the people of any State or Nation were acquainted with all the facts with regard to any problem, that they did not eventually by their majority vote solve such problem in a way that the thinking people of such community or Commonwealth might determine that it should be solved. * * * If I was convinced that the people of my State wanted the highways located for the interests of the few and not the many, I, for one, would waste no time in trying to secure economical location as I believe the people should have what they want, even if it costs more, when they have decided with all facts before them."

In a few isolated cases the Texas State Highway Department has, for safety purposes, undertaken to re-route certain highways. As a result, the officials and engineers have been charged with a tendency to isolate cities and towns. This is refuted by an examination of the facts. The State system now comprises about 20 000 miles of highways. As Mr. Gilchrist explained,

"When this system has been economically located or relocated the mileage will be about 19 000 miles, and not only will all the control points be reached, but, in addition, this system will reach directly, without material loss of distance, 97% of all towns * * * with a population exceeding 200.

"Included in this percentage are all towns that have a prospective main-line highway passing through their corporate limits or, in case of no corporation, within four blocks of the main business street."

Through Traffic.—A common viewpoint seems to be that a town loses a great volume of business when through traffic is shunted around. As to this, Mr. Gilchrist holds the view that:

"Doubtless certain classes of business establishments do profit to a certain degree by transient trade, but it would seem that a town depending to any degree on such trade would lack the foundation necessary for stable and continuous growth; that is, an established and a certain rural trade territory. * * * The solution of the highway or road problem for any community will not be reached until every village and town is either provided with a highway or with a durable connection to the highway, in order that the farmer or community trader might have access at all times not only to the State highway, but to the main business center of his trading point, and also to provide a way to come in or go out for the residents of such community.

"It would seem to be against sound economic judgment or good public policy to deflect the location of an important State highway, carrying 2 000, 3 000, or perhaps 5 000 vehicles per day, 2 or 3 miles out of the general course, in order to pass through the main business district of a community which would perhaps not originate more than 100 to 200 vehicles per day. * * * It is, or should be, the purpose of the engineer in selecting a route through a town, to relieve instead of create congestion."

Locating Roads for the Tourist.—He minimized the effect on local business, of by-passing the tourist traffic, stating:

"Neither the stability nor the growth of a community can depend upon the trade that will come from those who would ordinarily not stop at a place of business unless the highway happened to lead by the door. It would seem that the authorities in charge of the streets in a small town would prefer to reserve such streets for the citizenship of the community to use for parking their trucks or for stopping their produce wagons."

This attitude is often interpreted as a policy of building roads, not for the local people who pay the taxes, but for the tourist. The point of view changes rapidly, however, when a local taxpayer gets into his car and goes

out on the highways as a tourist. Because he pays *ad valorem* taxes and gas taxes, as well as other forms, he has just cause to expect that thoroughfare highways will lead as directly to his destination as possible, with side branches to near-by points along the line. From these considerations, Mr. Gilchrist reached the conclusion that State highways should be built from sources of revenue other than property taxes.

The Influence of Financing on Location.—State highway location will properly be considerably affected by the way in which it is financed, but no one property owner should claim special rights to traffic along a certain road on that account. When a hard surfaced road has been constructed on a certain street it should be continued and properly maintained, but the owners of business established on that route should not have the right to prevent the general traveling public from constructing other highways or by-passes as the need arises.

Mr. Gilchrist quoted at length from a recent editorial entitled "Getting Away from Antiquated Ideas"*. The idea expressed was impatience with engineers who design "roadways for a safe speed of 20 miles an hour while vehicles are operating at 60 miles an hour", urging instead that "new conditions must be met with new ideas."

In conclusion, he stated that highways should be built so that travelers (business men, farmers, or tourists) can reach their destinations with ease, directness, safety, and economy; that is,

"Highway engineers should take every opportunity to acquaint the public with facts regarding the economy of highway location and to lay before them for their inspection the ledger entries on the debit and credit side regarding the operation of a State highway. We should always have an eye for the future and if it is not possible to lay out a plan for the economical routing when first construction is obtained, the plan should be so broad as to anticipate the demands of the people, which are even now beginning to be felt, that in the location of State highways to serve centers of population an effort be made to relieve congestion and to promote safety instead of deliberately to create congestion and wink an eye at a dangerous condition just because by ignoring it some selfish interests will be served."

HIGHWAY LOCATION PROBLEMS

By A. R. Losh,† Assoc. M. Am. Soc. C. E.

One fundamental principle in the proper location of a road is that the road must be adequate for present and prospective needs. While the present needs are readily determined, future needs require the exercise of judgment and foresight. Mr. Losh defended the practice of improving existing roadways against a too ready willingness to relocate the road. Too often the cost of acquiring new and disposing of old right-of-way property is too great to make relocation feasible.

* *Highway Engineer and Contractor*, Vol. 20, April, 1929, p. 28.

† Dist. Engr., U. S. Bureau of Public Roads, Fort Worth, Tex.

Designing for Present or Future Conditions.—Discussing some of the findings of location practice in the past, he declared:

"There are still some who would consider the present highway building as primarily of local importance and for the immediate local needs, it being proposed at some future time to build a system of through highways on more direct lines. Such a line of reasoning has been followed to a considerable extent in the past with the following results:

"(*a*) The location of important through highways has been materially affected by purely local considerations.

"(*b*) Frequently local and through traffic are combined where a separation would be preferable.

"(*c*) When reconstruction for the through traffic is undertaken it is exceedingly difficult to make adequate relocation.

"(*d*) When the indirect line is used a number of years, development along it results in increased property values which may be seriously impaired by substantial changes in location.

"(*e*) Right-of-way changes resulting from relocations are usually more expensive than if made under the original construction, due to increased property values and damages to improvements.

"(*f*) The value of the old road in right of way, roadbed, surfacing, and structures, is a loss unless the road can be used advantageously for local purposes; but even in this case it must be considered that 1 mile of main highway costs as much as several miles of serviceable local roads and there is a loss in changing a main road to a local one.

"(*g*) The roadbed structure consolidated and compacted by years of use is lost for reconstruction purposes."

To follow this line of reasoning would result in engineers designing to meet the current needs of a community instead of leading it and preparing to meet conditions of the future.

Restrictions on Location.—Referring to another class of difficulties, Mr. Losh mentioned that:

"On all highway projects there are administrative controls or restrictions which determine at least the general location of the project, or the route of which it is a part. Thus, the termini of the project are predetermined and the intermediate location may or may not be fixed. These restrictions will occur under the following classifications:

"1.—Major control points of the Federal Aid System determined by the Secretary of Agriculture and the State Highway Departments and subject to modification by joint action.

"2.—State highways and control points designated by State Highway Commissions under legislative authority.

"3.—Control points for State highways fixed by legislation.

"4.—County or road district bond issues used for a portion of the construction cost may require a definite location to be followed."

The system of financing described in Restriction No. 4 has been responsible for most of the location difficulties in the Southwest during the past fifteen years. On the other hand, under Restrictions Nos. 1, 2, and 3, the tendency is to follow existing lines of improvement, establishing unimportant control conditions, and limiting the work of the locating engineers merely to details of design. But, continued Mr. Losh,

"Administratively, controls are necessary. They should be general in character and limited to the following conditions:

"(*a*) A town or city located on or near the general line of the route between adjacent control points of the route.

"(*b*) A town or city of sufficient importance to be a terminal point for the traffic passing between the adjacent controls.

"(*c*) A junction point, preferably near a town or city, of several highways for the distribution of traffic.

"(*d*) Topographical features based on engineering investigations.

"Least Cost.—Within the limitations established as controls the engineer, so far as practicable, should determine on an economic location, considering at their relative weights both through and local traffic. This selection will be the line which will provide the long run with the least cost, including construction, maintenance, and traffic operations. The shortest line will usually be the economic location for the reason that practically all items of cost except earthwork and structures vary directly with the length of the line. In a country of broken topography the cost of earthwork and structures may be much less on the longer line, but the other items of construction, and of maintenance and operation, will usually determine the location. Local traffic can generally be served cheaper by lower priced lateral spurs from the main line than by attempting to give direct service.

"One of the usual problems before the engineer is whether or not to swing the main line from its direct course in order to give highway service to an intermediate point, such as a small town or community. Theoretically, the problem can be solved by estimating the cost of construction, maintenance, and operation for each of the two possible layouts, that is, the main line direct with a lateral spur and the main line on indirect location through the intermediate point.

"A similar problem is that of combining two or three highways for some distance out from the city in order to reduce construction mileage. While economically desirable, frequently this involves the questionable practice of encouraging traffic congestion at junctions and terminals."

Effects of Topography and Grades.—Traffic on State highways usually is not local in character. In 1922, the average length of truck hauling in California, for instance, was 31 miles and that of passenger buses, 41 miles. Such through traffic is benefited most by being enabled to operate economically over the entire line rather than by being required to wind around through small communities with which it has almost no contact. Then, too,

"In working out the detail location further economies can be made by taking advantage of topographic conditions, soils, available local material, or railroad facilities for importing materials. Approximately 4 000 tons of materials are required per mile of 18-ft. concrete pavement. An increased dead haul of 1 mile will increase the cost of the road about $1 000 per mile. In addition, there will also be the cost of moving on and off the job; also, the hauling cost of structural materials. The use of curves of longer radii reduces the total length of line and the extent of surface widening."

Valley locations are often undesirable when they require costly, high embankments, etc., at overflow sections. Mr. Losh considers that the choice of stream crossings will depend on relative permanency of the banks, suitable foundations, required span lengths, etc., and that,

"Grades and curves adjacent to bridges should be reduced to the minimum. It is desirable to have a tangent of at least 150 ft. at each end of a major structure and also a light or flat grade. These conditions permit the driver to adjust his machine into the bridge alignment. Grades have probably received ample consideration in the past, or probably more than other features.

Considering the motor vehicles now in use on highways some sacrifice of grade can be made to improve alignment. Short grades as high as 8% can be taken readily by modern motor vehicles, but continuous grades of more than 6% are objectionable. Tangent profile grades are not particularly advantageous and do not justify increased expense. A rolling profile fitting the topography is less objectionable for night driving due to the headlight glare on tangent grades.

"*Urban Conditions.*—Grade separations at railroad intersections are desirable, but not always possible financially. Where grade crossings are adopted the line should be located so that future separation can be developed, and in all cases approach curves and grades should not restrict sight distances.

"Restricted areas in and near cities are frequently obstructions to direct highway location. This includes parks, golf courses, institutional reservations, cemeteries, railroad yards, and air ports. Generally, it is impracticable to pass through these areas, and the location must be carried around them. The location should give access to railroads and air ports, but be sufficiently removed to permit full operation of all facilities without congestion. A location directly adjacent to the track may be congested by trucks loading and unloading at the railroad and also by industrial spur-tracks being constructed across the highway. Location along the railroad is usually adopted because it is desirable from engineering considerations and for the further reason that right of way is less difficult to secure. It has the disadvantage, however, of providing for development on only one side of the highway, whereas if removed a quarter of a mile or more from the railroad property, both sides would be developed."

By-Passes.—This is an important matter. In fact, according to Mr. Losh,

"One of the major problems of the locating engineer is to provide suitable locations into, through, and around towns and cities. It is necessary to provide suitable connections into the street system of the city. It is highly desirable also to have a direct through connection even though it may be congested by city traffic, in order to take care of through traffic making a temporary stop in the city. A belt line or by-pass location is equally desirable for the through traffic to avoid the city congestion in reaching its destination and to permit the sorting or segregation of traffic for different locations.

"The desirable condition for the urban approach would be connections which would permit the distribution of traffic to:

"(*a*) The retail section;
"(*b*) The wholesale and industrial sections;
"(*c*) Railroads and air ports;
"(*d*) Several residential sections; and
"(*e*) To other highways by direct connections, outside the city proper."

Highways Through Cities.—In smaller cities the most insistent demand is that highways be carried through the principal thoroughfares of the city, preferably the main street, according to Mr. Losh, whereas larger cities are glad to avoid the congestion that follows such a location.

"In a great many cities, however, highways are taken directly through retail sections where the streets are congested with trolley cars, motor vehicles, and pedestrians. There are numerous street intersections, sharp curves, grade crossings, congestion due to parking, and other hazards of city traffic. The through traffic is delayed and local traffic congestion is increased with possibilities of accidents multiplied. Too many towns and cities are placing restrictions on through traffic by means of speed ordinances, stop signs, signals, and other regulations. These measures in most cases are necessary for the local traffic but the through traffic can better operate without them and outside the city. The following table gives information on several highways through cities of the Southwest:

"STATE HIGHWAYS IN CITIES.

City and highway.	Miles, in city.	Sharp Curves.		Railways. (All Classes).		Street Intersections.		Legal speed, in miles per hour.
		No.	Total degrees.	Number of tracks crossed.	Percentage of route used by street railway.	Control by signs, signals, police.	Total No.	
Dallas, Tex.:								
U. S. 80	8.5	12	735	19	12	15	89	10-20
U. S. 75	7.6	8	870	26	16	16	119	10-20
Fort Worth, Tex.:								
U. S. 80	10.9	6	390	9	51	12	123	12-18
U. S. 81	9.5	11	1 150	27	77	7	111	12-18
Houston, Tex.:								
U. S. 90	8.5	14	780	9	4	13	110	8-20
Little Rock, Ark.:								
U. S. 65	4.5	4	525	7	19	4	40	15-35
U. S. 167	3.3	2	180	7	6	4	35	15-35
U. S. 67	5.0	6	515	7	15	4	56	15-35
New Braunfels, Tex.:								
U. S. 81	3.2	9	635	1	0	....	30	20
Oklahoma City, Okla.:								
U. S. 77	6.2	6	510	21	9	6	70	25
U. S. 66	4.15	3	330	4	40	4	42	25
Purcell, Okla.:								
U. S. 77	1.2	4	360	4	0	1	13	....
San Antonio, Tex.:								
U. S. 81	10.0	14*	1 200*	8	28	13	84	10-20
U. S. 90	8.6	9	845	23	55	12	95	10-20
San Marcos, Tex.:								
U. S. 81	2.2	6	390	0	0	....	....	....
Tulsa, Okla.:								
U. S. 66	9.1	8	591	22	4	7	97	8-25
Waco, Tex.:								
U. S. 81	4.2	3	360	25	33	7	39	15-20

* Other routes give less curvature, but more traffic congestion.

Right of Way.—Mr. Losh predicted an inevitable increase in speed limits and advised highway engineers to consider this possibility in locations by providing greater sight distance on vertical and horizontal curves:

"To a great extent engineers have allowed right-of-way considerations to influence location. This is especially true in those sections where the section-line road system prevails. It is seldom that on rural highways more than 80 ft. of width is secured which amounts to approximately 9½ acres per mile. Considering the cost of State highways as about $30 000 per mile there could be a justified expense of $3 000 per acre for a proportional reduction in length of construction. This does not take into consideration any savings in maintenance and operation costs."

One evil that affects highway location, in Mr. Losh's opinion, is the tendency for abutting property owners to establish garages, service stations, concession stands, etc., close to the right of way, thus obscuring the view. The engineer should take advance steps to prevent this, for example:

"Acquire sufficient right of way for future development of the highway. Obtain extra widths at all road intersections and other strategic points where roadside enterprises would likely be established. In making acquisitions it will usually be possible to retain in the right of way the narrow strips, gores, and other small parcels of land cut off from the original tract. These parcels may be utilized for highway purposes in the future or for roadside planting and beautification. In any event, they are not available as sites for bill boards

and roadside establishments. Through areas of natural beauty either obtain in fee, or protect with long-term leases, several hundred feet of extra width on each side of the right of way. The interest of civic organizations and public-spirited citizens should be encouraged and their aid secured to obtain the desired ends."

Social Needs.—Last but not least in the location of highways are the considerations other than strictly commercial. The engineer is duty bound to provide access to social centers, scenic beauties, schools, churches, etc. Thus,

"As a function of the Government, every governmental unit should be connected finally with the country at large. * * * The larger unit of Government has this obligation to its constituent parts—the Federal to the State, the State to the county, and the county to its towns and communities.

"These considerations of schools, churches, Courts, and other conditions vitally affecting the citizenship of the Nation cannot be weighed in terms of economics or engineering, but must be taken under the 'general welfare' clause of the fundamental law."

EQUITABLE DISTRIBUTION OF MOTOR VEHICLE LICENSE FEES AND GASOLINE TAX

By O. E. Carr,* M. Am. Soc. C. E.

City Taxes for County Use.—In their present-day attitude taxing bodies seem, many times, to have "forgotten all equity" and to have been "actuated by the desire merely to get the money where it can be gotten most readily and with the least rupture." Mr. Carr went on to cite the fact that Tarrant and Dallas Counties, Texas, are about to spend about $8 000 000 on county highway construction which forms a part of the State highway work. The taxpayers of the two Cities of Dallas and Fort Worth are destined to pay about 80% of this expense. As he remarked:

"In every county in Texas, and practically all over the United States where there are large cities, the city dwellers pay their proper share of the support of various county offices, such as the sheriff's office, the constable's office, and that of the county board of supervisors. The work of these bodies is entirely done outside the corporate limits of any municipality."

Sketching briefly the evolution of thought relating to the financing of large projects, Mr. Carr called attention to the early predictions of some people that the great cost of the Panama Canal would wreck the Nation. It was the advent of the automobile that created the demand for paved highways, and with the attendant development came the need for various kinds of regulation. In most of the States, except Minnesota, the Legislatures have adopted a high licensing fee in order to finance the building and maintenance of roads.

When the development began in a general way, many city streets were already improved, whereas practically all country roads were not improved. Consequently, the income for highways was spent largely outside the immediate areas in which most of the taxes were collected. Quoting directly,

* City Mgr., Fort Worth, Tex.

"These conditions have been responsible for the fact that most of the States that charge a high license fee for registering automobiles, spend practically all these funds outside the corporate limits of the municipalities. At the same time, the major part of fees collected in the various States are paid in by those automobile owners who dwell within the corporate limits of our cities."

Evolution of Gas Tax.—The prospects for future expenditure in highway improvements are enormous. Despite the fact that Federal Aid Projects authorized at present are destined to cost more than twenty times that of the Panama Canal, the only question in the mind of the public is as to getting the work done fast enough.

Up to the present, the possible methods of financing seem to have been the primary consideration to the exclusion of studies to determine fair methods of distributing funds so collected. At first, license fees were the most popular means of charging the cost of highway improvements to the ones who used highways the most.

When the maximum possible revenue from this source was reached, Legislatures turned to the gas tax, which has now been established in every State except New York, Massachusetts, and Illinois.*

According to Mr. Carr,

"The theory of the gasoline tax is logical—it makes the individual who uses the highways, maintain them. It would seem that a fair way to apportion that maintenance would be on the basis of gasoline consumed by the automobiles. It is impossible, however, on this line of reasoning, to excuse the turning over to the educational funds of the State or to any other State funds, any part of moneys derived from the gasoline tax."

Distribution of Tax.—In Texas, a difficulty is encountered in the proper distribution of gas tax due to the State Constitution which provides that one-fourth of all moneys collected by the State from occupational taxes shall be used for educational purposes. One-third of all the tax resources of Texas are located in 4 of its 256 counties. Ten counties register 26% of all the automobiles in the State, and these counties, therefore, pay 36% of all money used for State highways. License fees are levied in two parts, one on the basis of horse-power and the other on the basis of weight. In 1928, these 10 counties paid to the State $4 187 340 in weight fees, and retained only $500 000 for county highways.

In various other States, Mr. Carr explained, there are situations analogous to that in Texas which render impossible an equitable distribution of the automobile and gasoline taxes. In many of the States the Legislature is either controlled by the rural representatives, or else the rural representatives are of sufficient number to secure for the rural communities more than a fair distribution of State funds. A few years ago it was a rare event when a State gave a municipality any part of the funds derived either from automobile licenses or from gasoline tax. State representatives are thus loyal to the district which they represent even though the situation is most obviously unfair.

* Since Mr. Carr's statement was made, New York and Massachusetts have adopted gasoline taxes; and Illinois is putting such a tax into effect August 1, 1929.—EDITOR.

Various Methods.—The first attempt to distribute funds equitably was made by Ohio in 1919. At present, 50% of all license fees collected in that State are retained by the municipal corporation in which the automobiles are owned. The remainder is available for the maintenance and repair of State highways. The gasoline tax is likewise distributed, 30% to the municipality, 25% to the counties, and 45% to the State.

Other States have not worked out such equitable distribution systems, but many of them have progressed appreciably. The status of eleven States in this respect is listed by Mr. Carr as follows:

"Alabama: Gives 20% of the motor vehicle license fees to the city in which the automobile is owned.

"Indiana: Gives 0.25 cent for each gallon of gasoline sold in the State to the cities, distributed on a population basis.

"Iowa: $200 per mile is given the larger cities for the maintenance of State or primary roads, and in cities of 2 500 population and less, the State constructs and maintains the State and primary roads.

"Kansas: Cities of the first and second class are given $250 for each mile or fraction of a mile of State road street. In cities of the third class the county commissioners have the option of giving the cities $250 for each mile or fraction of a mile of State road, or they may maintain the streets.

"Louisiana: An extra tax of 1 cent per gal. is collected for each gallon of gasoline sold in the City of New Orleans, this sum being placed in the paving fund.

"Maryland: 1.7 cents is paid the City of Baltimore for each gallon of gasoline sold in the State of Maryland. The money is used for the building and maintenance of streets and for providing grade separations.

"Michigan: Gives each city $2 000 per mile for each mile of State highway in the city limits.

"Oklahoma: The cities are given 15% of the motor vehicle license fees collected in the city limits.

"Virginia: Constructs and maintains the State road in cities of less than 2 500 population, having houses more than 200 ft. apart.

"Washington: Gives each city $500 per mile for each mile of State highway in the city limits.

"Wisconsin: Gives the larger cities $300 to $500 per mile for the maintenance of State highway streets in the city limits and from $25 to $200 per mile for city streets not a portion of the State or county road system."

Use on City Streets.—In 1927, the total income for gasoline taxes in the United States was $258 838 813, of which about 60% was paid by city dwellers. Nevertheless, only $2 000 000 of this amount was used for the construction and maintenance of city streets. It seems clear to Mr. Carr,

"That the authorities should appropriate as much money for the maintenance of the city streets forming a part of the highway development as is spent on the State highways outside the city. Furthermore, it would seem absolutely inexcusable on the basis of equity for the State to fail to do these things. On the other hand, it would be entirely unreasonable and selfish for any city to expect to retain all the gasoline tax and all the automobile license money collected within the municipality for the sole use of improving and maintaining its city streets."

In conclusion, he recommended the system of distribution used in Ohio as the most equitable in the United States at present. In his opinion it is not possible to establish definite rules for an equitable distribution of these funds. The solution for each case will depend on local conditions which will vary from time to time. He called attention to tables that are available at the U. S. Bureau of Public Roads:

"Which give the total amount collected in several States during the year ending July 1, 1928, from automobile license fees and also the gasoline tax in the several States; also the amount derived from the registration fee in 1927, as well as the amount derived on account of the automobile gasoline tax."

DISCUSSION

DISTRIBUTION OF HIGHWAY FUNDS BETWEEN THE STATE AND ITS MINOR POLITICAL SUB-DIVISIONS

By C. E. Myers,* M. Am. Soc. C. E.

In this discussion, read by J. C. Carpenter, M. Am. Soc. C. E., Mr. Myers welcomed an interchange of views on what he considered "the most controversial subject uppermost in the minds of highway engineers". There is a great question as to whether State Highway Departments should have exclusive use of revenue from license fees and gas-tax funds, or whether parts should be distributed to county highway projects in areas of unusually dense population.

Practice in Philadelphia, Pa.—The City of Philadelphia, for instance, occupies less than 0.1% of the entire area of Pennsylvania, while one-fifth of the total population lives within the county lines. Not a single foot of State highways is located within this area. To quote Mr. Myers directly:

"Philadelphia has 1 881 miles of improved streets of varying widths and 154 miles of unimproved streets, making a total of 2 035 miles. If this were estimated on a State highway basis, that is, all streets a uniform width of 18 ft., the city would have a total mileage of 2 886 miles. As against this, the State Highway System, none of which is within the city limits of Philadelphia, has a mileage of 13 200, and received last year (1928) from motor license fees and gas tax funds approximately $52 000 000. From these figures, city highway engineers feel that the city does not get, in receiving $602 500, anything like the amount it should, when consideration is given to the city's mileage system with its much heavier traffic and also to its extremely dense population. Until 1922, no State highway funds at all were made available for the City of Philadelphia. Since that time a certain amount has been available each year, as shown in the tabulation that follows.

"It will be noted that until 1928 at no time has the city furnished funds for the Bureau of Highways to as great an extent as it did in 1921, before it had any State funds at all. There has happened in Philadelphia what I fear may happen in all too many other minor political subdivisions if State money is distributed instead of being spent by the State; that is, that instead of

* Director, Dept. of City Transit, Philadelphia, Pa.

making more money available for the different highways within the cities or counties, the city or county appropriating authorities will simply appropriate less money for highway work from their current revenue because of the State aid money they are receiving."

Year.	Appropriated from State Gas Fund to City of Philadelphia.	Spent from city funds—current revenue, loan funds, and assessments.	Total spent by Bureau of Highways—all sources.
1921	.....	$ 8 531 000.46	$ 8 531 100.46
1922	$115 000	7 624 184.95	7 739 184.95
1923	86 000	6 062 114.33	6 148 114.33
1924	618 000	6 510 367.12	7 128 367.12
1925	284 000	7 839 551.36	8 123 551.36
1926	641 000	8 333 995.45	8 975 795.45
1927	984 400	7 827 134.16	8 811 534.16
1928	602 500	10 018 739.38	10 621 239.38

Proposed Limitations.—Specifically, Mr. Myers recommended that:

1.—Funds from State revenue should be allocated for use in cities or counties only when the local authority had appropriated as much or more than its average appropriation during the preceding five years.

2.—The allocation should be based on mileage and population within the cities or counties.

3.—The State should have the right to designate the type of work to be done and the routes to be selected with the funds it supplies to a city or county.

4.—The money should be given to the minor political subdivisions and should be actually spent by them.

Under the present system the tendency is for a city to receive more and more funds from the State and decrease its own appropriation proportionately so that State funds do not actually go toward supplementary improvements within the city.

GENERAL DISCUSSION

Experience in Iowa.—Discussing the paper by Mr. Carr from the viewpoint of conditions in Iowa, President Marston stated that the main object, for some time to come, should be "to secure the construction of a State-wide system of roads", such that traffic lines will remain open through the year. In Iowa, said President Marston,

"It certainly would put the completion of that program back if any material proportion of the recepits from either automobile license fees or gasoline taxes were simply turned over to local municipal bodies.

"We established a gasoline tax there a few years ago, a certain fraction of which, for example, went to the townships. Now, it is my judgment that they might as well have left that tax off for all the good it has done in benefiting the highways of the State of Iowa; and I think that, in connection with such funds as are spent inside the city limits, it is wise to have the State officials assume real authority over the streets on which those expenditures are to be made, and to see that they are made in such a way as to give us connections

from our main lines of roads or State roads into the heart of the city. I do think that the expenditure inside the city limits should be made by the local municipal bodies and not by the outside authorities; but that a great deal of authority should be retained in the hands of the State body."

Road Funds in Their Entirety.—Mr. Losh agreed with the argument for the use of automobile license fees and gasoline taxes, as presented by Mr. Carr. Of the gasoline tax collected in Pennsylvania during 1928, 5% was used for purposes other than roads. In his opinion, there should be:

"An equitable distribution of road funds made to the several classes of roads. This distribution should contemplate the entire fund, and not merely the gasoline tax and automobile tax constituting only 35% of the highway work. Bear in mind, aside from gasoline and auto taxes, we have *ad valorem*, personal property, and municipal taxes on motor vehicles; we also have road taxes spread by counties and cities.

"Consider that the entire rural road fund of 1927 is $1 400 000 000, and the sources of income are as follows: 18.6% from bonds; 35.2% from gasoline tax on automobiles; 40.7% from *ad valorem* taxes; then if we are going to consider the distribution of the highway funds, I think we should take all these funds."

In closing the discussion, Mr. Carr stated as regards *ad valorem* taxes on automobiles, collected by the taxing districts:

"That tax is very difficult to collect and not more than 50% probably is collected by the cities and counties, generally. Then, as a rule, no vehicle tax is levied by the cities on automobiles."

Cities Autonomous.—He took issue with Mr. Myers as to the proper distribution of funds for improvements to city streets:

"The city authorities are the best judges of what money to spend on city streets. City taxes and other taxes are high. If these funds come in from another source, it is very natural—and I think perfectly right and proper—for city authorities to reduce the taxes, having this other money to spend on its streets, for this purpose, providing the roads and streets are kept up in a reasonable condition."

Furthermore,

"It may be, as is true in many cities, that a city bonds itself very heavily in order to make certain improvements, paving streets, etc. That would put the big expenditure on these propositions for a certain time; after that the taxpayers pay the bill in the shape of charges for their bonds and interest fund. Not one city has done that, so it is necessary to spend less money in the future for a certain year or number of years, and they should not be penalized on the allowance from the State on account of having had foresight enough to make that expenditure when it was needed."

The meetings of the Highway Division and the Construction Division were held jointly, the papers of the former being read and discussed first. The reading of the paper presented as part of the program of the Construction Division was followed by the showing of numerous slides on concrete road-building plants.

CONCRETE ROAD BUILDING PLANT AND ITS OPERATION

By Tyree L. Bell, Jr.,* Assoc. M. Am. Soc. C. E.

In his preliminary remarks, Mr. Bell outlined the scope of his paper, stating that it was restricted to concrete pouring methods and concrete machines and plants. He explained that he had not "in any manner tried to go into the mechanics or economics of the operation of a plant, but had merely outlined what is orthodox in concrete construction with reference to equipment and methods, giving some experiences and some suggestions for its operation."

This interesting and valuable paper expressed in detail many features which make for economy in road building everywhere, with especial reference to experience in Texas. The keynote was expressed as follows:

"The contractor's object is to do good work, and, at the same time, obtain a profit from his work. His opportunity for obtaining a profit is largely dependent upon the machinery which he uses and the efficiency of the labor which he employs. Therefore, the selection of machinery, its assemblage into a plant and the efficiency of its employment, are of paramount importance to the contractor.

"Efficiency is no accident, nor is it to be had purely through energy and enthusiasm. Efficiency, ordinarily, results only from careful systematic planning, from the determination of some specific course by which a desired result can be had, and, then, from setting about with tireless persistence to follow that course. It is the result of careful planning, hard work, and good management."

Value of Good Equipment.—Modern development in the field of plant management has resulted in many improvements for the highway contractor. Development has been so vast in many cases that road builders could not keep abreast with it. They have had scant time to study many of the economic phases of proper choice of machines for use in their construction plants. Accordingly,

"The contractors' task has been to select, from a mass of available good equipment, that which would best fit his proposed scheme of operation, to perfect the use of the machines until they would give regularly all the service of which they are capable, and to co-ordinate their functions into a smoothly running plant."

Since it must be admitted that a contractor's business is to make money, he must be more interested in the return of production than in the creation of outstanding production records. Mr. Bell advanced the opinion that record runs reflect very little profit except in the case of an occasional spurt for advertising purposes. The maximum number of square yards is most likely to result from the program of steady, uninterrupted production. In insuring this uniformity,

"Paving should be viewed as a series of operations, each performed at a rate which yields production in harmony with all the others. The paving mixer is the key producer, the pacemaker, for the entire plant. The other

* Gen. Supt., F. P. McElwrath Co., Corsicana, Tex.

machines and the crew must be assembled around the basic idea that their inter-dependent production efforts must be balanced and co-ordinated around the paver. In the sequence of operations which must be performed, each element must be scheduled to do its part a little in excess of the capacity of the pace-making mixer, so that short delays need not affect the output of the mixer."

In addition to the larger machines used on road-building work, the contractor will find it worth while to study the economies in various special hand tools and other innovations that have been developed to fill some particular need. If the equipment problem has been settled, the contractor must consider the proper choice of crew. As modern methods requiring power-driven machines have so largely replaced manual labor, the labor that is still used must be of a higher general quality than was formerly the case. In describing the plant layout for work in Texas, Mr. Bell feels that it typifies standard practice in other places in the United States.

Rough Grading.—Mr. Bell made no effort to indicate that rough grading could be summarized briefly into anything resembling a standard, because the work to be done varied so greatly. In his own words,

"The specifications and the design of the embankment, the topography of the country, the nature of the soil, and the season for doing the work, all require consideration in determining how the rough grading should be done.

"The methods of grading, in themselves, have not shown the same degree of improvement as has the surfacing. The fresnoes and wheel scrapers of the South and Southwest still keep the price of dirt work at such a low figure that it is not sufficiently attractive to interest the efforts of alert contractors. The jobs are let in small units and light yardage."

In some other sections of the country, where the extent of rough grading on heavy embankments has resulted in a more substantial and better developed basis of procedure than in Texas, the steam shovel, the drag-line, elevating grader, and new crawler wagon seem to prove quite satisfactory.

Material Yard.—On highway work it is very important that an adequate supply of proportioned materials be available for transportation to the mixing plant at all times. The choice of a plant will depend largely on the relative certainty and ease of delivery of material as needed. On some large jobs it is feasible to provide vast storage spaces in which material can be stocked during the winter when construction is at an ebb, so that, in the spring, work can be resumed at once by utilizing this material. Mr. Bell, however, was principally interested in the construction of roads that required materials to be delivered direct to the job as needed, and the subject of the paper was confined within these limits. In detail,

"The equipment required for such a plant is a crane and overhead bins, equipped with measuring devices, and, for auxiliary service, special trackage and a cement warehouse. The crane is of the crawler type, full revolving and handling a clam-shell bucket, ranging in size from ¾ yd. to 1¼ yd. If the capacity of the crane and the skill of the operator are sufficient, all the unloading and proportioning sufficient for the full requirements of a 7-sack mixer can be accomplished during daylight hours with the one crane. Quite frequently, however, it is found that a night operator is necessary, in order to unload out of cars into storage enough of the materials to free the regular operator from a part of the car unloading, that he might concentrate on

keeping the bins full by digging largely from the stock piles. Greatly increased operating speed can be obtained by loading directly from the stock piles to the bins, rather than from tightly packed cars to the bins."

Construction Track.—The choice of track layout will depend on the efficiency of the switching service provided by the railroad company. A crane, some bins, and a stock pile are ordinarily set near the middle of an unloading track. The length of track necessary is twice the combined length of a string of cars long enough to hold the usual amount of material needed for one day's work. The reserve stock piles should be provided as a source of aggregate supply if regular shipments of aggregate are delayed in transit. Storage of cement should be governed by the same considerations, according to Mr. Bell. In addition, a space should be available for the emergency storage of cement that comes in while the job is shut down.

Bins.—Bins are available in various sizes and designs. In general, they should be rigid and strong enough to withstand unusual and erratic loading, and at the same time they should be light enough so that they can be moved as economically as possible. Mr. Bell approved of the present tendency toward measurement of aggregate by weight. In fact, he stated that it,

"Is now an accepted and established practice. Certain Highway Departments specify weight measurements; others favor them. Weight measurement provides for greater accuracy in the proportioning of aggregates. The bulking of sand, due to the moisture content, and the varying voidage of the coarse aggregate, seriously disrupts the accuracy of volume proportioning. Several manufacturers offer weighing batchers that are entirely satisfactory in operation and comparatively inexpensive. Changes in the sizes of the batchers can be accurately made with ease and with absolutely no delay."

The use of bulk cement with the help of bucket elevators, screen conveyors, and proper weighing devices offers an opportunity for economy.

Hauling.—Hauling by means of truck is still the most widely used method of transporting materials on road-building projects. In Mr. Bell's opinion, for "hauling one, two, or three batches of proportioned materials, the truck is the most flexible and adaptable transportation means that we have." At present, the tendency is for large multiple-batch trucks to displace the lighter single-batch one. Most contractors prefer to hire their trucks and pay for them on a batch-mile basis. Some still prefer to operate their own fleet, thereby assuring constancy of service at all times. That is,

"If trucks are used, there should be a sufficient number of them employed to insure there being at the mixer always enough trucks standing by, to remove any possibility of the mixer waiting for a batch. The number necessary is dependent upon the speed of the truck, the length of haul, and the time required at the material yard for batching. Any money spent by the contractor in the maintenance of roads, which will assist the trucks to maintain their full speed, without having to slow down for bad holes or barriers, will greatly re-act to his benefit."

In some cases turn-tables have been found economical as part of the equipment designed to save time in turning trucks. In certain localities, narrow-gauge railways are an advantage, but this type of plant "has not been accepted as readily as seemed inevitable at one time" on highway work.

The first cost of narrow-gauge equipment is high, the cost of transporting the equipment from job to job, and the time delay in setting up such equipment, are likewise high as compared with the more mobile trucks. Nevertheless, trains can be organized and run on more dependable schedules after the construction line has been established, so that Mr. Bell recommends a combination of narrow-gauge railroads and trucks, when the length of haul is long enough to warrant it.

"*Water Supply.*—It has been most aptly said that 'the water supply is the life of a paving job, and the pump is the heart of the water supply'. When the pump stops, the work stops. Second only to delays of management, the Bureau of Public Roads' Time Studies show that more time is lost on paving outfits from the failure of water supply, than from any other individual cause."

Modern development in pumps has made available several that are especially designed for road building. Some of them will supply "from 80 to 120 gal. per min., against a 500-lb. pressure." Mr. Bell stated from his experience:

"That from 40 000 to 60 000 gal. of water per day are sometimes required during the hottest part of a Texas summer. Although such demands can be met by a single standard road pump, if in good condition, by doing some of the curing at night, it is better practice to provide two independent pumps.

"It has been found especially advantageous, where the location of the water supply allows such procedure, in using two pumps, to keep one pump ahead of the mixer, in order that its supply and pressure will remain undiminished by the demands of curing, while the other pump is left behind to furnish curing water. Pumping from different sources and working against each other, the two pumps, in that way, should maintain a constant pressure throughout the entire line, and neither mixing nor curing interferes with the other. Then, in the event of a shutdown, either pump could temporarily supply the combined needs of the entire job."

Enough water pressure and pipe capacity should be available so that if one batch of concrete has been taken from the mixer there will be enough water available at once for the next batch. This feature involves only simple hydraulics, but pumping equipment and construction practice have now become so standardized that "except in unusual cases, no particular thought is given to hydraulic study". Black iron pipe is used everywhere for carrying the water from the pump along the road. It should be laid in a ditch so that leakage water will naturally drain away from the job. Instead of a 2-in. pipe, contractors are now using 2½ and 3-in. pipe, figuring that the decrease in friction as well as the greater capacity of the pipe will offset the added purchase cost and transportation charges.

Relief valves at the pump and at the mixer are important as safety devices in the event of sudden stoppage. Mr. Bell recommended that outlets be provided every 500 ft. for the connection of a mixer hose 300 ft. long. In fact,

"A large number of contractors use two mixer hoses that one may always be furnishing water to the mixer, while the other is being moved ahead. Hose for the mixer in the very best obtainable quality, in 1½ to 2-in. sizes, should be provided."

Form Setting and Fine Grading.—To a large extent, the tools and methods used in setting forms are the same everywhere in this country. Quoting, again,

"It has long been recognized that to secure smooth riding pavement, it is necessary to have strong, rigid, and well-set forms. While several makes of forms are on the market, the designs of all types are sufficiently similar to standardize their setting operation. Forms should be supplied in an amount at least twice that of a maximum day's run. Economy in forms is 'penny-wise' economy. The farther in advance the forms are kept, the more economically can the sub-grade crew work. The forms are a gauge and templet for both subgrading and finishing. Consequently, contractors have all along sought forms that have offered, with ease of installation, the greatest strength and rigidity. After the forms have been set, the sub-grade is ordinarily scarified thoroughly, from form to form, to a depth of 0.2 ft. A subgrading machine, frequently equipped with an auxiliary scarifier, is next pulled along the forms, making an initial cut and leaving the sub-grade slightly above the net grade. The excavated material is spread uniformly along the shoulders. The sub-grade should then be lightly rolled with a power roller, weighing preferably from 3 to 5 tons. When the sub-grade has been rolled to a uniform compactness, the subgrader should again be run over it, a cut being made to the exact neat lines of the section, and the excavated material wasted. Pulled behind the paver, another subgrading machine, called a trail grader or sub-grade planer, can be used to excellent advantage."

Mixing.—The 7-bag batch size of mixing machine is now most generally used, thanks to the standardization efforts of the Equipment Manufacturers' Association; but any paving mixer ordinarily purchased by a contractor will be found to be a dependable machine. According to Mr. Bell,

"The mixer is the pace-maker for the entire operation. It is the bottle-neck of the production. All the other elements of the plant should be synchronized and organized to assist in securing from the mixer its maximum capacity in output. The cycles of full operation of the different machines, under a 1-min. mix specification, will range from 1 min. and 10 sec., to 1 min. and 25 sec. Mechanical devices are now available to control the cycle of the operation, but, as yet, are not generally used, for, as a rule, an alert operator can get out of the mixer its maximum capacity, provided the other elements of the plant function properly. To develop the full capacity of the mixer, the operator must be able to make it hoist, mix, dump, and travel simultaneously."

The acceptance of the water-cement-ratio theory is credited to the establishment of greater accuracy in the control of water, and devices for this purpose have been considerably improved in late years.

Finishing.—Mr. Bell advocated the use of mechanical finishing machines in preference to hand methods, and stated that,

"A straight and rigid form line and a concrete of uniform consistency are the prime essentials of a good finish. The tools and machines to be used, whatever they may be, cannot get the results of which they are capable without such assistance.

"Specifications and the whims of certain engineers sometimes require special hand methods of finishing behind the machine. Such finishing is too dependent upon personal skill. A level, unyielding form line, and a properly functioning machine, in the speaker's opinion, provide a uniformity and mechanical exactness of surface finish not at all possible where hand tools are used.

"*Shoulders.*—The demand for greater refinement in the dressing of the shoulders has occasioned the development of machines the finished work of which should please the most exacting. Such machines, running on the pavement, and with blades gauged by the edge of the slab, cut the shoulder and slope

to the exact section of the plans. Both, however, are dependent on an adequate supply of loose material on top of the shoulders."

Chemicals are being used more and more to aid in curing concrete pavement, but the old method of an earth covering sprinkled with water, or that of "ponding", is still being used in the majority of cases.

Benefits of Improved Methods.—In conclusion, Mr. Bell stated that,

"Better roads and cheaper roads are being built. Such improvement has been brought about by the speeding up of operations and the consequent increased efficiency and reduced labor and overhead costs. If you will reflect that, within a period of five years, the output of paving plants has been doubled, that the labor cost has been cut in half, and that the plant expense has not increased, you can appreciate the wonderful strides that have been made in concrete road building."

Replying to a question by Chairman W. J. Barney, Mr. Bell stated that in his opinion the day of steam for mixing plants on road work is past. From recent investigations and papers, it has been quite definitely established that concrete can be hauled for an hour or two without detriment.

SANITARY ENGINEERING DIVISION

APRIL 25, 1929—9:30 A. M. TO 12:30 P. M.

Before proceeding to the regular program of the meeting, Russell Suter, M. Am. Soc. C. E., Chairman, introduced Karl Imhoff, M. Am. Soc. C. E., as a guest of the Division. Immediately thereafter the first speaker was presented.

FORT WORTH SEWAGE DISPOSAL PLANT'S IMPROVEMENT

By W. S. Mahlie,* Esq.

Early Sanitation.—As a result of developments begun during the Eighties, Fort Worth now has about 550 miles of sanitary sewers which serve 85% of the city's population, according to Mr. Mahlie. The establishment of two large packing houses in 1903 has added greatly to the contamination of the Trinity River. After numerous suits had been filed in protest against conditions in that stream, the State Legislature enacted a law on July 1, 1913, making it illegal "to pollute watercourses and other public bodies of water, and giving cities of 50 000 or over three years to make such provisions as necessary." Two expert consultants were retained in 1914 and empowered to make a sanitary survey of the city. They recommended the establishment of Imhoff tank and sprinkling filter treatment. They also recommended that the packing houses establish a means of treating their wastes before it entered the city sewerage system.

In 1919 a bond issue of $725 000 was voted for building a sewage disposal plant. Legal developments, however, prevented the sale of the bonds under the conditions proposed, and the work was never begun. In 1921 other consultants were retained, and they recommended activated sludge treatment. The recommendation was over-ruled by the City Commission, which felt that the Imhoff tank sprinkling filter method was best, and the plant with a capacity of 7 500 000 gal. per day was designed on this basis. The estimated population equivalent was 125 000.

New Imhoff Plant.—In due time, this plant was constructed and was put in operation on April 24, 1924. It is located 3½ miles east of Fort Worth on the Trinity River. As described by Mr. Mahlie:

* Chemist in Chg. of Water and Sewage Purification, Fort Worth, Tex.

"It consisted of bar screens, grit chambers, electric-driven pumping plant, Imhoff tanks, sprinkling filters, final sedimentation tanks of the Dorr type, chlorinators, sludge drying beds, and all the necessary appurtenances.

"The outstanding features of the plant were as follows: The Imhoff tanks were rectangular in shape, 96 ft. long, with a total depth of 25 ft., and consisted of a double-flow channel, each half being 16 ft. wide. The gas vents covered 25% of the surface area. The detention period based on complete displacement flow was 1.8 hours. The sludge capacity was estimated at 151 000 cu. ft., or 1.2 cu. ft. per capita. The filter beds, three in number, had a total area of 3.18 acres, with 8.2 ft. depth of stone. Sewage was applied through Taylor square nozzles spaced at 12 ft. centers on squares, not staggered. The two final sedimentation tanks were 80 ft. in diameter. Dorr clarifiers provided 1½ hours detention. The sludge bed area was 1.4 acres."

Treatment of Packing-House Wastes.—Complying with the advice of the first consultants, the packing houses prepared in 1904 to provide a preliminary system of treating the waste from their plants by constructing septic tanks, contact beds, and sprinkling filters. This plant was found unsatisfactory, and, in 1905, they were advised by a consulting engineer to install a septic tank, a rough filter built of large rock, a subsidence basin, and a final gravel filter. No action was taken on these recommendations. However,

"In 1916, an experimental activated sludge plant was built. The results obtained were good, but the cost was too high and, consequently, the plant was never built. When the city started building its disposal plant in 1923 and 1924, the packers decided to install a screening plant, and as the work progressed they (the packers) decided to add Dorr clarifiers. Their plant was completed and put in operation in February, 1925."

New Studies.—An analysis of sewage in Fort Worth shows that the City Disposal Plant produced a good, well-modified effluent for about 10 months. The addition of waste from the packing houses increased the normal strength of the city sewage considerably. This sum led to the conviction that an enlargement of the plant was urgently needed. Accordingly, in the summer of 1927, expert consultants were again retained to report on the situation. Their report was made in November, 1927, recommending a plant of suitable size to treat 16 000 000 gal. daily. It was stated that,

"Three methods of procedure were outlined:

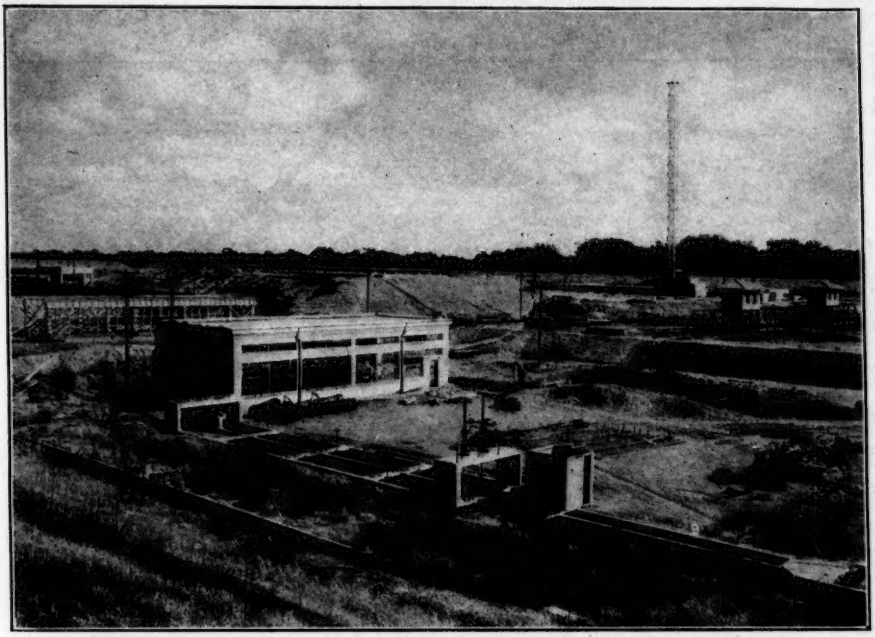
"(1) To convert the existing plant into a pre-aeration type of plant, similar to that at Decatur, Ill.

"(2) To construct an activated sludge plant of about 11 000 000 gal. capacity, and operate it as a separate plant, in addition to the existing plant.

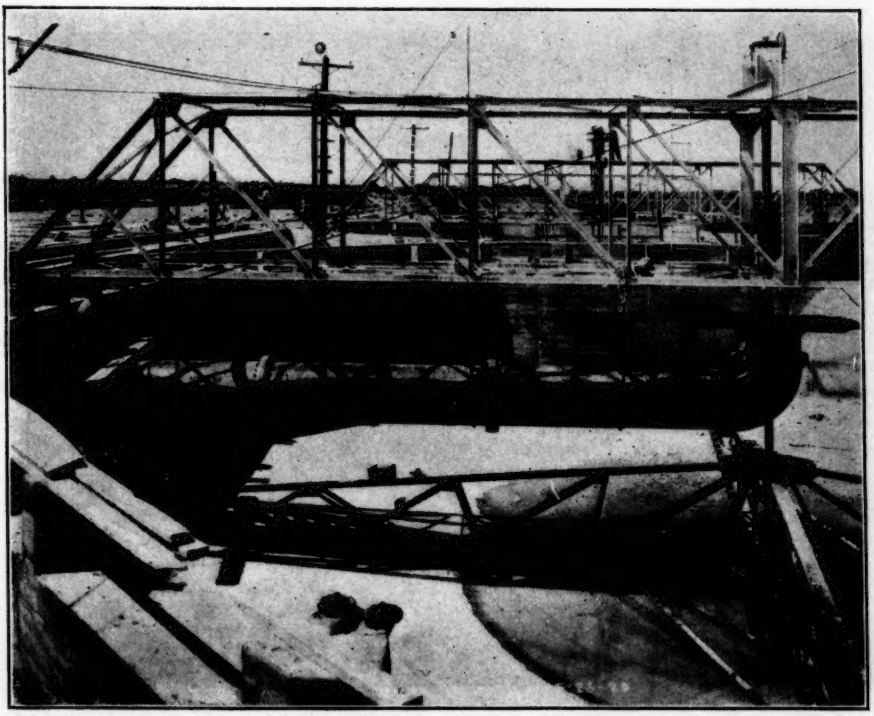
"(3) To enlarge the present plant along the same lines as then existing, namely, Imhoff tank and sprinkling filters."

It was felt that "preliminary sedimentation, with separate sludge digestion, followed by a short period of aeration and subsequent application to sprinkling filters, would be best suited to the conditions".

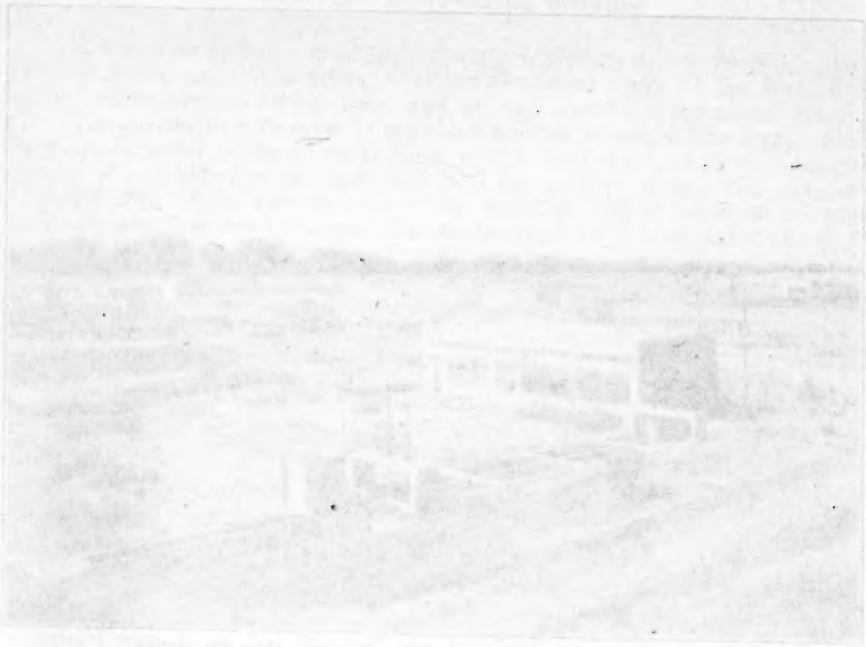
Plant Now Building.—The plan submitted was followed in the design, with re-aeration of sludge as an added feature. The work begun in August, 1928, is now (April, 1929) nearing completion. According to Mr. Mahlie,

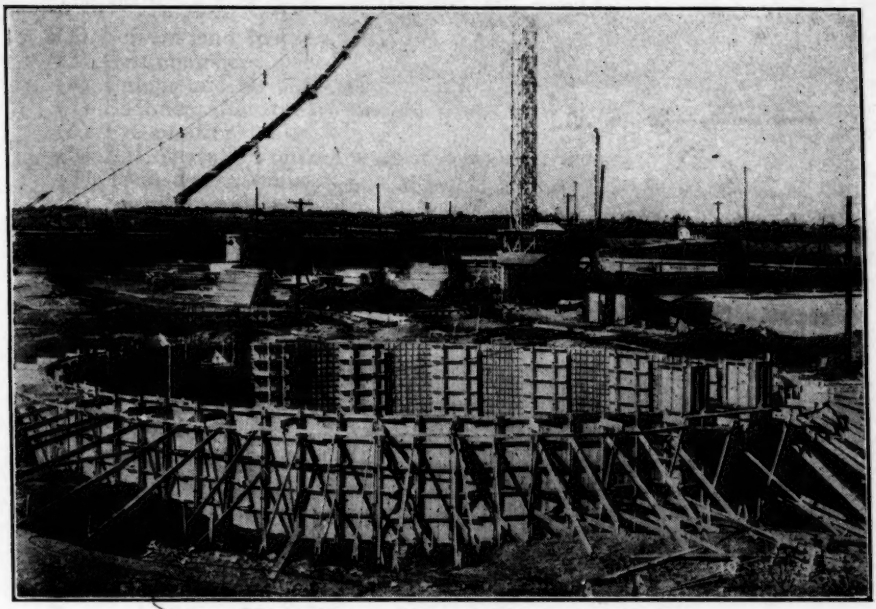


FORT WORTH PLANT: GENERAL VIEW, SHOWING GRIT CHAMBERS AND PUMPING STATION.

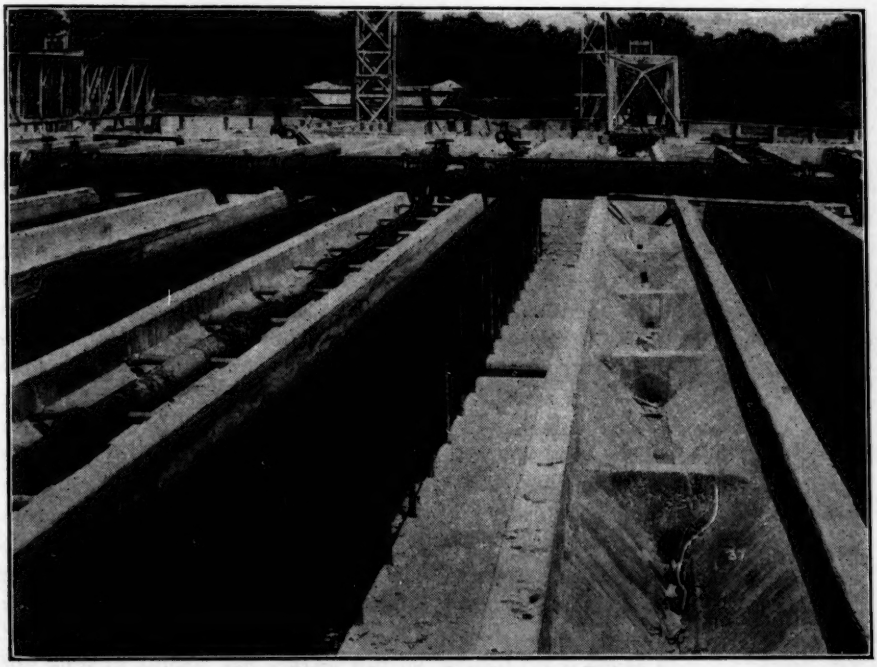


FORT WORTH PLANT: VIEW OF CLARIFIER FOR SETTLING PARTLY ACTIVATED SLUDGE.

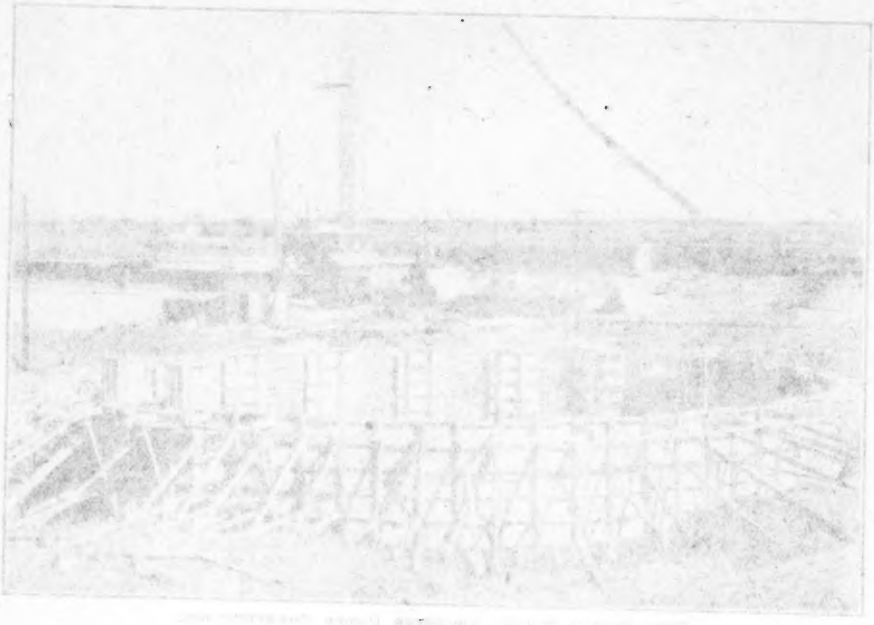


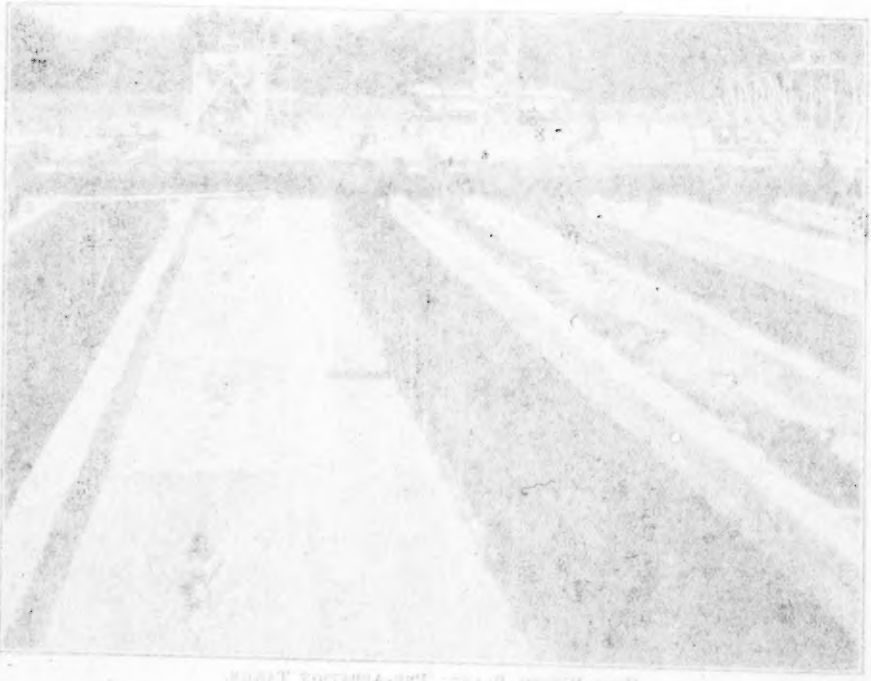


FORT WORTH PLANT: DIGESTER UNDER CONSTRUCTION.



FORT WORTH PLANT: PRE-AERATION TANKS.





"The plant as now being constructed will provide:

"(1) Screens and incinerator.
(2) Grit chambers.
(3) Pumps and air-blowers.
(4) Sedimentation tanks—Imhoff type.
(5) Pre-aeration tanks.
(6) Sedimentation tanks for aerated sewage.
(7) Twin dosing tanks.
(8) Sprinkling filters.
(9) Final clarifiers.
(10) Sludge digestion tanks equipped for heating and gas recovery.
(11) Sludge lagoons."

In an alternate plan, provision was made for generating electric power by means of gas engine-driven generators utilizing the gas from the digestion tanks. It was decided, however, to collect more exact data regarding quality, character, and rate of production of the gas before spending the $100 000 required. Then,

"The improved plant will have a designed capacity for an estimated flow of 16 000 000 gal. per day, representing 225 000 population expected to be reached in 1942. The maximum daily flow expected in 1942 is 32 000 000 gal., and the maximum hourly flow is at the rate of 45 000 000 gal. daily. The present flow is estimated at 11 500 000 per day, of which 9 000 000 gal. is domestic sewage and 2 500 000 gal. is partly treated packing-house waste."

Proposed Operation.—Following this description, Mr. Mahlie gave a detailed account of the operation of the plant as now constructed. The raw sewage will be delivered to the screen and the incinerator house through a 54-in. line. The present 1-in. bar screens are being changed for a mechanically cleaned bar screen with ½-in. openings. The raking mechanism is float-controlled, and is driven by an electric motor through suitable gear reducers. An incinerator with a capacity of 40 cu. ft. of wet screenings in 10 hours will be fired with gas obtained from the sludge digestion tanks. From the screens the sewage passes through a 42 by 18-in. Venturi meter to the grit chambers. The detention time is 51 sec., and the velocity, 1.8 ft. per sec. From this point the sewage will be pumped to preliminary settling tanks by six pumps, having a total capacity of approximately 45 000 000 gal. daily. These pumps operate against 28½ to 29½ ft. of head. By using Imhoff tanks for preliminary sedimentation, it will be possible to draw the fresh solids to the digestion tank each day, or, if necessary, allow them to become partly digested before pumping. In the latter case, however, the gases formed in the first stages of digestion will not be recovered because provision has not been made for covering the gas vents. From the Imhoff tank, the sewage will pass to the aeration tank, consisting of 26 channels, 145 ft. long, 10 ft. wide, and 11 ft. deep. These are arranged in five units of five channels each, and one unit consisting of one channel is re-aerating the sludge. The central unit of the five is arranged so that all or a part of it can be used for re-aeration in addition to the one channel especially provided for that purpose. Then,

"When handling 16 000 000 gal. per day, the designed capacity, and returning 25% of sludge, the sewage will be aerated for 3 hours and the sludge for ½ hour."

About 9 300 cu. ft. of air will be supplied by three centrifugal blowers, which will be diffused through "Filtros" plates in the bottom of the channels. The ratio of plate to surface area is 1 to 6, and air will be supplied at the rate of 0.5 cu. ft. per gal. The aerated sludge will then pass to three secondary settling tanks which are 75 ft. in diameter, 15.67 ft. deep at the center, and 13.33 ft. deep at the edge. Sewage enters at the center through a 36-in. pipe passing through the side wall and overflowing an upturned special ell in the exact center, then passing under an octagonal baffle surrounding the entire unit. According to Mr. Mahlie:

"The settled sewage will flow toward the periphery and pass over adjustable weirs extending completely around the tank, into proper collecting channels and then to the sprinkling filters. The tanks provide for about 1.6 hours' detention."

The sludge is then pumped from these tanks to the re-aeration unit. The total area of all the beds will be 4.24 acres, and sewage is to be applied at approximately 4 000 000 gal. daily per acre. From the sprinkling filter the sewage will flow to the final clarifiers, where provision is made for detention of 1 hour and 23 min. Then it goes through the chlorinating chamber to the effluent outfall, where a flap gate is provided to check back-flow in case the river rises. The total cost of these improvements will be about $780 000. An analysis of sewage at Fort Worth is as follows:

FORT WORTH SEWAGE ANALYSIS
(Values are in parts per million)

	Fort Worth, domestic, raw. (1)	Fort Worth, domestic, treated.* (2)	Packing house waste as delivered to city mains. (3)	Mixed sewage now received at Sewage Disposal Plant. (4)	Final effluent, mixed. (5)
Total solids	1 172	826	6 015	2 162	1 795
Suspended solids	341	38	395	361	58
Non-settleable solids	133	23	308	138	29
Settleable solids by volume, in centimeters per liter	7.4	0.5	0.8	6 5	.85
Nitrogen as free ammonia	19.0	7.6	46.0	28.5	10.6
Nitrogen as total organic nitrogen	21.0	....	171.0	32.7	10.0
Nitrogen as nitrite and nitrate	.....	8.2	.....	.....	5.0
Oxygen consumed	104	26	537	138	48
1-day bio-chemical oxygen demand	168	15	992	283	24
5-day bio-chemical oxygen demand	372	36	1 603	520	42
Dissolved oxygen	.....	3.5	.....	....	1.6
Alkalinity to methyl orange	316	264	888	473	....
Chloride	175	....	1 766	413	.....
Methylene blue stability	.....	86%	.....	.....	76%
Average flow, in million gallons per day (estimated)	9.0	....	2.5	.....	.....
†Population equivalent	163 200	....	195 300	.....	.....

* Column (2) refers to first 10 months of operation.

† Based on 0.17 (5-day) bio-chemical oxygen demand per capita.

DISCUSSION

IMPROVEMENT OF THE PLANT

By S. W. Freese,* Assoc. M. Am. Soc. C. E.

The prepared discussion presented by Mr. Freese, is pertinent as well as concise, and is quoted herein in its entirety:

"There are several features mentioned in Mr. Mahlie's paper which are possibly of special interest.

"1.—The mixed Fort Worth domestic sewage and packing house sewage is comparatively strong with 2 162 parts per million total solids and a 5-day bio-chemical oxygen demand of 520 parts per million.

"2.—The present plant, that is, before enlargement, is of the Imhoff tank-sprinkling filter type.

"3.—The size of the present plant is being trebled by:

"(*a*) The introduction between the Imhoff tanks and the sprinkling filter of activated sludge units which provide ½ cu. ft. of air per gal. of sewage in 3½ hours. The sludge is settled out at a rate of 1 333 gal. per sq. ft. per day.

"(*b*) An increase in Imhoff tank, capacity of 25%, giving a preliminary settling period of 1 hour.

"(*c*) An increase in sprinkling filter area of 33⅓%, such that the aerated sewage will be taken at the rate of 8 500 lb., 5-day bio-chemical oxygen demand, per acre per day.

"(*d*) An increase of 200% in sludge digestion capacity.

"4.—A mixing chamber is provided for mixing the raw sewage and the return sludge.

"5.—The aeration tanks are 10 ft. deep by 10 ft. wide in cross-section, such that the tanks can be adapted to some form of mechanical aeration if found to be more economical.

"6.—The secondary clarifiers are octagonal with central baffled inlet and with peripheral weir as outlets.

"7.—Two-stage sludge digestion is provided, the first stage in the Imhoff tanks and the second in separate sludge digestion tanks. The separate sludge digestion tanks are equipped for sludge mixing, scum breaking, sludge heating, and gas collection.

"8.—Provision is made for measuring the total gas production, the amount used by the screening incinerator and for heating the sludge. It is further planned to install 800 h.p. of gas engine-driven generators to use the gas from the digestion tanks."

In the layout no particular parts were left for the extension of the plant, according to Mr. Freese. The city owns land inside the levees that is about twice the area occupied by the new plant and also owns three or four times as much land behind the levees. Split tile is used for the bottoms of the filters. The decision to continue the use of split tile was made because:

"When the new sprinkling filters were designed, the question came up as to whether to use split tile, as previously, or one of the new patent bottoms which seemed to work so nicely; and to find out whether the present split tile was working satisfactorily, we dug down into the old tank and found

* Cons. Engr., Hawley & Freese, Fort Worth, Tex.

they were clean, that there was no disintegration, and evidently no reason for discontinuing their use."

Replying to F. C. Scobey, M. Am. Soc. C. E., as to what difficulties are being encountered in maintaining the orifices of the Venturi meter and what necessity there was for periodic cleaning, Mr. Mahlie explained that provision is made for a constant back-flow into the orifices to keep them clean.

OPERATING RESULTS AT DECATUR, ILL.

By Samuel A. Greeley,* M. Am. Soc. C. E.

Since the experiences at the plant in Decatur, Ill., affected parts of the design of the Fort Worth plant, Mr. Greeley presented some of the later operating results observed at Decatur, taking the month of September, 1928, as reasonably typical. The Decatur plant was designed for a population of about 60 000, and in addition to the tanks and accessories, it included 3 acres of sprinkling filters. The presence of a cornstarch factory placed a heavy burden on the plant. In part, Mr. Greeley said:

"The industrial waste from the starch works overloaded the plant much as had the packing houses at Fort Worth. Aeration tanks were provided in addition. During the month of September the flow averaged 11.37 gal. per day, all delivered to 3 acres of sprinkling filters, making a rate of nearly 4 000 000 gal. per acre per day. Prior to the installation of the aeration tanks, that rate was seldom more than 1 500 000 gal. per acre per day."

The bio-chemical oxygen demand for September was 172 parts per million, which is equal to that required for a population of 96 000. The effluent of the aeration tank filters reduced the oxygen demand 43.2 parts. This was then placed on the sprinkling filters and passed through the final settlement tank so as to reduce the demand 91.9 parts. The demand for the final settling tanks was 14 parts per million.

The electric power used for pre-aeration was 93 850 kw-hr., which was at a rate of about 0.42 cu. ft. per gal. of sewage. Speaking of overflow conditions at Decatur, Mr. Greeley stated that:

"As is the case with many plants handling sewage from combined sewers, it is necessary at times of high flow to overflow at the plant small volumes of the sewage. * * * This overflow is now measured and a determination made of the strength and is computed back to the population equivalent. The average amount by-passed was 230 000 gal. per day; which, figured from its oxygen demand, gave a population equivalent of 1 900. This, of course, went untreated into the river."

GENERAL DISCUSSION

Should a separate supply of gas from a city line be used instead of the gas from digestion? How is that being handled at Fort Worth? What is the grading of the filtering media? What arrangement is made for removing

* (Pearse, Greeley, and Hansen), Chicago, Ill.

the fines before applying and for the development of the fines during the time the stone is being put in the bed? These questions by F. M. Veatch, Assoc. M. Am. Soc. C. E., were discussed by the authors and various other speakers in turn. There is no auxiliary gas line from Fort Worth to the plant. The pilot light is maintained only by gas from the digestion. The stone is graded according to the same specifications as that for the old plant; that is, from 1½ to 2½ in., before being put on the beds. The contractor runs his cars right out over the bed and unloads the stone from the cars on the ground in that way.

According to L. L. Winans, Assoc. M. Am. Soc. C. E., an attempt was first made to wash the rock, but the contractor had more success by using air pressure and a blower. The dust is blown out of the rock at the crusher plant and the material is then delivered in carloads.

The practice at Wichita Falls, Tex., was outlined by H. R. F. Helland, M. Am. Soc. C. E. After considerable difficulty, good clean stone was secured from Oklahoma and Texas. As stated by Mr. Helland,

"It was placed by shovel from tracks running over the stone. Immediately after being placed the fire hose was turned on it and it was washed that way. Since that time, as far as I know, there has been very little fines coming out of the plant."

UNUSUAL FEATURES OF SAN ANTONIO, TEXAS, SEWAGE DISPOSAL PLANT

By H. R. F. Helland,* M. Am. Soc. C. E.

Preliminary Considerations.—Since 1901 the disposal of all sewage from San Antonio has been done under contract with an irrigation company. Sewage was conducted to an artificial lake called Mitchell's Lake, about ten miles south of San Antonio. The waters from the lake were used for irrigating approximately 4 000 acres of land. In recent years the sewage flow has become too heavy for this arrangement, and the city has employed engineers to make necessary reports and draw plans for a more adequate treatment. Accordingly,

"Preliminary to entering into the design of the plant, sewage flow measurements were carried on for a period of ten months. These measurements were made in a section of concrete flume through which the sewage flowed to Mitchell's Lake. The average flow for ten months was 16 400 000 gal. per day, with a maximum flow of 28 400 000 gal. per day, at which rate some sewage was spilling over the sides of the channel. This maximum flow occurred during a period of storm. From September 10 to November 30, 1928, a sample of sewage was taken each day, the time of taking such sample being an hour later each day for the period. Analyses of these samples gave an average of suspended solids of 260 parts per million. Tests on these samples showed 68.4% volatile matter and 31.6% ash, giving 0.163 lb. per capita per day of dry solids contained in the sewage."

Activated Sludge Plant.—The present population of San Antonio is estimated at 285 000, of which probably 220 000 contribute to the load of the

* Associate Engr., Hawley & Freese, San Antonio, Tex.

sewerage system. The sewage flow is at present 16 400 000 gal. daily, or 75 gal. per capita per day. Describing the proposed new plant, Mr. Helland stated:

"The San Antonio sewage treatment works is to be an activated sludge type of plant, designed to handle 30 000 000 gal. of sewage per day, thus providing for an increase of approximately 85% in the present sewage flow. It is estimated that this will serve a contributing population of 400 000 people."

The new plant will be located about five miles south of the city. The sewage will be conducted to the plant through a 76-in. concrete outfall sewer. The total flow will be measured by means of a 60 by 36-in. Venturi meter, equipped with automatic register-indicator-recorder. There are to be two channels for the flow of sewage in the screen house, in one of which will be installed a mechanical rotary screen with ½-in. openings between bars, and in the second channel, a plain bar screen with openings of similar width. The incinerator in the screen house will utilize the gas from the digestion tanks as fuel. As the sewage goes to the grit chambers, its velocity is reduced to approximately 1 ft. per sec., in order to promote sedimentation. There are three channels in the grit chamber, two of which will suffice for passing the entire flow while the third is being cleaned. Two square primary clarification tanks which are next in line, have an average water depth of 11 ft. These are designed on a basis of a 30-min. sedimentation period, with an expected removal of 42% of suspended solids. Skimming devices and scum drains are to be installed at one corner of each tank. Next in line are the aeration tanks, which are 20 ft. wide, with a 15-ft. depth of sewage. There are three rows of filtros plates along one side of each channel, the ratio of plate area to tank area being approximately 1 to 7. The sewage flows back and forth through three channels, giving a total length of aeration channel of about 450 ft.

Route of Flow.—In regard to the flow, Mr. Helland stated that,

"Sewage enters into the aeration units through submerged 36 by 36-in. sluice-gates and is taken out over adjustable weirs into the outlet channel across the lower end of the aeration tanks. The total area covered by the aeration tanks is 400 by 194 ft. These aeration tanks are designed for 5-hour aeration periods, the rate of aeration being 1.5 cu. ft. of air per gal. of sewage."

The final clarifier tanks are 90 ft. in diameter, with a 15-ft. water depth. They are designed for a 2-hour sedimentation period, figured on a basis of design flow plus 20% return sludge for 1 600 gal. per sq. ft. of tank area daily. There will be four sludge digestion tanks, each 75 ft. in diameter, and 25 ft. deep. They will be equipped with Dorr stirrers and scum breakers. The roof will be concrete, supported by I-beams suspended from the supporting truss. These tanks provide for a 45-day storage of sludge. Digested sludge will be pumped from the digestion tanks to two sludge lagoons, each 200 by 200 ft. in area. The installation will be as follows:

"Except for the raw sludge pumps, all equipment will be installed in a pumping station 30 by 75 ft. in dimension. All pumps will be installed on the lower floor, which is below the water level in all units of the plant. Suction and discharge pipes are carried in a pipe gallery between the final clarification

tanks and in a gallery along the end of the aeration tanks. The location of pumping units at this level tended to allow the use of lower head pumps than would have otherwise have been required."

* * * *

"The treated effluent from the plant will discharge into the present earth channel which carries the sewage to Mitchell's Lake. From this lake, the water will either flow into the Medina and San Antonio Rivers, or may be used for irrigation. There is a possibility that an irrigation district will be worked out, using the effluent from this plant for irrigation purposes."

* * * *

"The plant will be well lighted. Flood lights will be installed at the screen house and on the main pumping station, in addition to which lights will be placed on the clarifier tanks and on the digestion tanks."

Declaring that the plant is unique in that its location was extremely fortunate as regards construction features, Mr. Helland described its location, as follows:

"It is to be in a small valley, across which the sewage is now carried in a wooden flume. The elevation of the floor of the valley is such that practically no excavation will be required for constructing the aeration tanks. The maximum depth of excavation which will be necessary for the digestors, will be 15 ft. The plant is very compact, there being no waste of space between the units where unnecessary. The total area covered by the plant proper, exclusive of sludge lagoons, is 500 by 425 ft. The grit chambers, screen house, and Venturi meter chamber are situated 300 to 500 ft. from the plant proper for construction reasons, due to the topography of the ground. The total loss of head through the plant is less than 1 ft. The sludge digestion tanks furnish an unusual feature in design for the southern part of the United States, in that they are equipped with heating coils for maintaining an optimum temperature for the digesting sludge. Future extensions can be made on ground to the east."

The lump-sum contract price for the construction of this plant was $654 000, exclusive of the equipment for the primary clarifier tanks, final clarifier tanks, sludge digestors, mechanical screen, and the air compressors, which were bought by the City for $150 000.

DISCUSSION

OTHER UNUSUAL FEATURES

By Marvin C. Nichols,* Assoc. M. Am. Soc. C. E.

"The development of adequate sewage treatment works in Texas prior to the last few years," stated Mr. Nichols, "has been much slower than in other sections of the country. Although Texas is the largest State in the Union, having an area of 266 000 sq. miles, its population in 1920 was only 18 persons per sq. mile. This may be compared with Illinois with a population of 115 per sq. mile, New York, 227, Ohio, 141, Massachusetts, 500, Pennsylvania, 192, and New Jersey, 383. In 1920, Texas had 30 cities with a population greater than 10 000. Of these thirty cities, approximately one-half have made extensive

* Associate Engr., Hawley & Freese, Fort Worth, Tex.

improvements to the sewage disposal works during the last ten years. Owing to the low density of population, the scattered cities, and climatic conditions, the development of sewage treatment works has been neglected to some extent in Texas. This condition, however, is rapidly changing. Cities that a few years ago discharged raw sewage into the rivers, are now building modern treatment works."

Popularity of Activated Sludge Treatment.—Fort Worth constructed an Imhoff tank sprinkling plant in 1924, which is now being doubled in capacity. Modern activated sludge plants have been constructed in Waco, Houston, Amarillo, and San Marcos. The latter was one of the first activated sludge plants constructed in the United States (1916). For a number of years following 1917, the plant in Houston was the largest activated sludge plant in the world.

"The trend of sewage treatment works in Texas", remarked Mr. Nichols, "is toward some form of activated sludge treatment for the larger cities. The San Antonio sewage treatment plant, an activated sludge plant, will be unusual in several respects: First, the plant is designed to be flexible in operation; second, the plant is designed for separate sludge digestion, a departure in Texas practice; third, the initial cost of the plant per million gallons of sewage treated is extremely low; fourth, the plant is located and designed to permit of later additional capacity east of the plant now being built."

"Flexibility and Capacity.—Due to the rapid increase in population of Texas cities, and to possible improvements in methods of treating sewage, one of the essential prerequisites of a treatment works is that the plant be flexible. The San Antonio plant is so designed that (1) all sewage may be by-passed, or (2) by-passed around any unit; (3) the time of aeration may be varied; (4) the raw sewage may be aerated; (5) the activated sludge may be re-aerated; (6) the detention period in final clarifiers may be varied by using all or part of the tanks; (7) additional units may be built later for any of the various operating units."

The design features of the San Antonio plant were reviewed briefly by Mr. Nichols to show the basis of computing the volumes to be handled.

The sludge lagoons mentioned by Mr. Helland will have a capacity of approximately 560 000 cu. ft. If this is added to the 395 000 cu. ft. volume found for the four 75-ft. digestion tanks, the capacity is 2.3 cu. ft. per capita, or 15.4 cu. ft. per lb. of solids per day. These factors are low, admitted Mr. Nichols, but it is believed that, with proper heating, the sludge capacity will be sufficient.

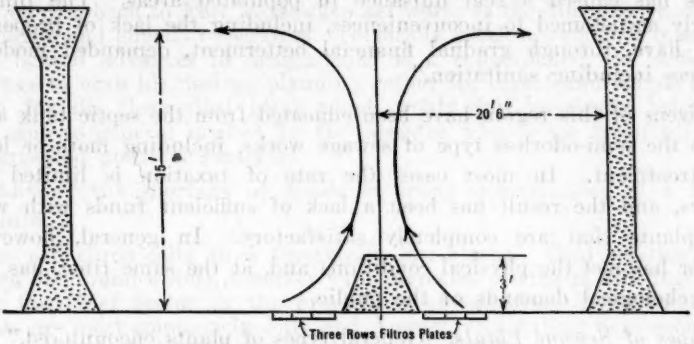
Various Features.—In regard to other features, Mr. Nichols declared:

"It will be particularly interesting to observe the results obtained in the separate sludge digestion tanks. The flexibility of the plant will permit of additional digestion tanks should they be required. Due to the location of the plant, no immediate complaint can result from lagooning the digested sludge.

"The effluent to the final clarifier tanks is brought in through a 42-in. pipe at the center of the tank. Around the pipe is set a wooden baffle suspended from the truss. It is believed that this radial flow outward to the adjustable weirs around the rim of the tank will provide a more uniform flow than is ordinarily obtained with a straight flowing through of the sewage.

"For experimental purposes, the upper portion of one of the walls separating the aeration channels has been omitted. This channel will be approximately 40 ft. wide. Filtros plates are installed at the center on both sides

of the low baffle wall. It is believed that it will be possible to produce spiral flow in both directions from this low wall. If such is found practicable, considerable saving can be effected by the elimination of the separating wall."



BAFFLE WALL TO PRODUCE SPIRAL FLOW

The San Antonio plant will cost approximately $900 000. On the basis of design capacity of 30 000 000 gal. per day, the cost per 1 000 000 gal. daily is $30 000. This is somewhat less than the usual cost for plants of this character.

SIZE OF TANKS

Samuel A. Greeley, M. Am. Soc. C. E., commented on the omission of the central wall, thus providing a tank 40 ft. wide instead of 20 ft. Quoting him in part:

"One of the units of one of the recently completed North Side tanks in Chicago was built 35 ft. wide, the tank being 15 ft. deep. This unit has been in operation more or less regularly since October, 1928, and while operating results are not extensive enough to permit speaking with any finality, nevertheless the results as measured by the nitrate and oxygen demand appear to be quite satisfactory as compared to reduction with the same sewage in tanks only 20 ft. wide.

"Partly as a result of this and other studies a unit of the New York Ward's Island plant has been set up with a tank a little more than 22 ft. in width, with a depth of 15 ft."

EXPERIENCES IN DESIGN AND OPERATION OF PLANTS IN EAST TEXAS

By H. L. Thackwell,* Assoc. M. Am. Soc. C. E.

Local Conditions.—Geographically, East Texas has an area about the size of the State of Illinois, with a moderately dense well-scattered population. According to Mr. Thackwell, it

"Has a definite sanitary problem to face in regard to sewage disposal. Temperatures range from + 14 to + 110° Fahr. Precipitation ranges from

* Cons. and Const. Engr., Jacksonville, Tex.

35 to 50 in. per annum. The streams, swollen in floods, reach out and inundate the muddy forested banks, retreating in dry weather to tortuous sluggish channels. Oxygen deprivation in polluted streams during the hot dry summer months has caused a real nuisance in populated areas. The inhabitants, formerly accustomed to inconveniences, including the lack of proper sewage works, have, through gradual financial betterment, demanded modern conveniences including sanitation."

Citizens of this region have been educated from the septic tank and cesspool to the semi-odorless type of sewage works, including more or less complete treatment. In most cases the rate of taxation is limited by city charters, and the result has been a lack of sufficient funds with which to build plants that are completely satisfactory. In general, however, the designer has met the physical conditions and, at the same time, has satisfied the psychological demands of the public.

"Types of Sewage Plants.—General types of plants encountered," as mentioned by Mr. Thackwell, "range from simple Imhoff tanks to complete plants of various units, including separate sludge digestion, de-gasification with heating, glass-covered sludge beds, contact aerators, trickling filters, humus tanks, Dunbar beds, final aerators, pre- and final chlorinators, etc.

"In order to economize, the structures have been made with thin walls reinforced to satisfy actual stresses. No money is usually spent on constructing monumental works. Structures, however, are not flimsy, but are substantially built."

Separate sludge digestion, de-gasification, partial pressure digestion, reaction treatment, dosing of sludge, decantation, air agitation, heating, and glass covering have been adopted in various plants. It was found, for instance, according to Mr. Thackwell, that it was not necessary to use heat, except during September and January. He continued:

"Open air beds are popular in connection with Imhoff tanks. Decanted liquor from sewage beds is filtered through sand. Settling tanks which are efficient in removing solids to separate sludge tanks, are usually hand-operated without mechanical appliances other than valves and sluice-gates. Contact aerators, as used at Jacksonville, are adaptable for partial treatment, and add materially to the flexibility of any plant having other units.

"Trickling filters are more or less of the conventional type, although central dosing tanks, infiltration tunnels, channel-sluicing devices, and banked sprays, have been used with success. Dosing tanks are usually single, operated with a trapped or trapless siphon. Aeration of influent into dosing tanks has proved beneficial, likewise chlorination at this point has had a salutary effect in regard to odor control. Humus tanks should follow trickling filters but, for economy's sake, are frequently omitted. Two storied humus tanks having a rate controller and scum-removing devices, as well as a separately housed pumping station which enables digested humus to be pumped either to sludge beds or lagoons without consequent fly breeding or other nuisance, have been operated successfully at Jacksonville. Activated sludge plants have been designed, but not recently built. Laundry wastes at Jacksonville are to be treated in a small activated sludge plant followed by sand filters."

Handicaps.—One influence that continually works against the feasibility of adopting the most modern equipment, Mr. Thackwell remarked, is the difficulty of securing competent operators:

"All ventures in disposal plants requiring chemical adjustments or mechanical control have added to the cost and difficulties of operation. The most serious drawback to East Texas disposal works is the difficulty in obtaining intelligent and effective operators. Generally speaking, efficient operation lags well behind advances in design and, as a consequence, the designing engineer has to curb his desires, planning rather for direct and simple operation than for the theoretical ideal which would fall hopelessly below the standard, due to general neglect by the operator who is too frequently just an inexperienced political protégé."

In addition, the shortage of funds in many communities is a factor. For instance,

"Cities are chronically in a financially low status. Cold weather, with infiltration of ground-waters, together with increases of stream diluting water, has been the chief factor in the prevention of lawsuits. Injunctions and complaints are most common in July and August, and during September conditions have generally improved to such an extent that promises for improvements are usually sufficient to prevent lawsuits."

For an efficient control of odor in a small plant, the site should be at least 1 500 ft. from the nearest house on a tract of land at least 10 acres in area. Further, Mr. Thackwell thinks that,

"Digestion tanks should be de-gasified, sludge-dried under glass, and the settled effluent given a partial oxydizing treatment by contact aerators or coarse-grained filters, this effluent then being treated by continuous feeding on Dunbar beds arranged in three units. Final aeration and chlorination should be adopted if indicated.

"Such treatment will leave no indication of polluting organisms in the stream below the plant, and freedom from pollution, together with the absence of odors and fly breeding, will be the practical test of a good plant."

DISCUSSION

OTHER EXPERIENCES

By H. N. Roberts,* M. Am. Soc. C. E.

Importance of Operation.—The area under consideration could boast very few sewage disposal plants of any kind about seven years ago. There were some septic tanks and a very few Imhoff tanks. Mr. Roberts gave credit to Mr. Thackwell for his part in improving sewage disposal conditions in East Texas. He concurred in the opinion as to the difficulty of securing competent operators, and constant careful supervision of the plant. He quoted a remark made by the State Sanitary Engineer:

"'Sometimes I think that the best location for the sewage disposal plant is on the Public Square, then it would get some attention and be kept operating properly'."

Furthermore, Mr. Roberts recommended that the designing engineer increase his fee to include the cost of several months expert supervision of the

* Cons. Engr., Lubbock, Tex.

plant after it has been put in operation. He should either furnish the operator himself during this period, or be given the authority to engage and discharge the plant operator. According to Mr. Roberts,

"The present system is indeed most wasteful; considerable money is spent on a plant. Then the plant is promptly forgotten; until it becomes such a nuisance that a lawsuit is quite often the result. Upon investigation it is found that the plant has been so neglected that it requires a complete overhauling at considerable expense."

Experience in West Texas.—In view of this condition, Mr. Roberts has followed the practice of installing the very simplest type of plant that will take care of the situation. In West Texas, which has an average annual rainfall of 22 in., advantage is taken of the sandy loam soil to establish systems by which the settled sewage is used for the irrigation of crops. In 1926, the City of Lubbock began to apply settled sewage to 88 acres of land outside the city.

"*Crops.*—The crops raised are as follows:

Plat.	Crop.	Acres.	Condition.
1	Red Top cane	10	Excellent
2	Cotton	4	Average
3	Cotton	12	Very good
4	Popcorn	3	Very good
5	Milo maize	9	Very good
6	Corn	9	Excellent
7	Cotton	25	Very good
8	Watermelon	2	Good
9	Corn	3	Excellent
10	Beans	1	Good
11	Sudan grass	2	Good
Total		80	"

With no losses, the daily flow was equivalent to 4.7 in. per month on the cultivated land. With 40% evaporation loss and seepage, there would be left 3 in. per month going to the crops. Adding to this the amount of rainfall and deducting evaporation from the soil and transpiration, the net residual moisture is 3 in. going into the soil each month.

"Experience at Lubbock has indicated," Mr. Roberts stated, "that conditions and results would be better if this residual to the soil were about one-half this. This would mean that 1 acre of this land would take care of 2 200 gal. of settled sewage per day.

"A thorough inspection of the farm has not been made recently, but it is understood that after three years' operation it is operating successfully, without nuisance, and with no apparent change in the soil from its original condition."

GENERAL DISCUSSION

In answer to a question as to whether the irrigation previously described breeds flies, J. N. Chester, M. Am. Soc. C. E., was told that there was no indication of flies, odors, or any other nuisances.

Chemical Disintegration of Stone.—N. T. Veatch, Jr. M. Am. Soc. C. E., inquired as to the chemical and mechanical theory that accounted for the disintegration of stone under the action of sulfur. Mr. Thackwell replied that he accepted the generally adopted theory that sulfates in water will have a tendency to develop hydrogen sulfide gas. In his experience,

"This gas, especially when sprayed out with sprinkling filter sprays, has had the effect of disintegrating concrete in a number of installations I have examined personally. * * * I have found that by the use of various paints properly applied there seems to be no disintegration."

DISTRIBUTION OF COSTS OF SEWERAGE SYSTEMS

By W. S. Tanner,* Assoc. M. Am. Soc. C. E.

A Difficult Problem.—The scope of this paper covered the original cost of construction, and the maintenance and operation of the collection as well as of the disposal systems. To arrive at an equitable distribution, according to Mr. Tanner, is a difficult problem. For instance,

"In the past, funds for maintenance and operation of the collection and disposal system in Dallas have been provided by an *ad valorem* tax. This is manifestly unjust, since many residents of the city having inadequate, or no, sewage facilities are required to share in the cost, not only of new construction, but also of maintenance and operation of both collection and disposal systems. Others, who are satisfactorily served, have paid in *ad valorem* taxes far more than their just share, in order to increase the size of the disposal plant, or to extend present lines, when these increases and extensions did not directly or materially benefit them."

In considering the distribution of costs of maintenance and operation, Mr. Tanner continued, both the quantity of sewage and the degree of its pollution should be considered. Similarly, with a Water Department, the rate might be based simply on the quantity furnished, or on the purity, if several grades were available.

Sanitary Sewer Districts.—This condition is being remedied by the organization of sanitary sewer districts and the Legislature has recently passed an Enabling Act for the formation of such districts in Texas.

Describing briefly the means by which these districts are organized, Mr. Tanner stated:

"After the organization of a district has been approved, construction of the collection system should be financed on a basis of assessments of benefits and damages. This permits equitable consideration of existing collection systems within the proposed district, which is impossible when extensions are financed by the old *ad valorem* tax method."

New Method of Distribution.—He then described in detail his own solution for equitable distribution of costs. A flow chart for a certain line of sewers indicates the varying discharges throughout the twenty-four hours. From this, the minimum flow, which occurs in the early morning hours, gives the

* With San. Sewer Dept., City of Dallas, Dallas, Tex.

infiltration into the system, the remainder being domestic flow. Similar charts are available for several lines.

On each line an enumeration is made of the various sanitary fixtures. The rates for each fixture are given an algebraic symbol so that on each line an equation may be made up involving the infiltration plus the unknown amounts for the types of fixtures and equalling the flow on that particular line. Using as many equations as there are unknowns, and solving, gives the required rates.

A similar method is used to determine the degree of impurity for each of the fixtures. For example, by applying the rates already found, the total flow for each type is known on a given line. The equation then consists of the summation of productions obtained by multiplying the flow and the algebraic treatment for the unknown bio-chemical oxygen demand rate, the whole summation being equal to the bio-chemical oxygen demand for that particular line, in parts per million. Thus, the bio-chemical oxygen demand for each type of fixtures is obtained.

The presentation of costs to be charged for the various fixtures is obtained by averaging the fractions found for the number of units contributing to the system, and the relative bio-chemical oxygen demand for each type.

Subdivision of Costs.—The application of these results, as described by Mr. Tanner, consisted in fixing for each item of cost—whether collection system or disposal system—a logical allocation of the expenses according to the reasonable use and purpose. For example, the maintenance of the collection system is subdivided solely on the basis of quantity. Thus, the reasonable rates are fixed on a logical basis for this complicated engineering structure.

DISCUSSION

COST OF MAINTENANCE AND OPERATION

By R. G. Hemphill,* Assoc. M. Am. Soc. C. E.

The discussion by Mr. Hemphill consisted of a detailed description of twenty-four reasons why all civic organizations of Dallas County, Texas, should support the Act pending in the State Legislature permitting the formation of sanitary districts. He stated that Michigan, Ohio, Wisconsin, Minnesota, Illinois, Missouri, California, and New Jersey, and possibly other States, have already passed similar legislation. Briefly, the reasons given are, as follows:

1.—Increased valuation of outlying sections of a city, due to adequate facilities for connecting with the city sewage system.

2.—The effect of restoring the original beauty of the Trinity River.

3.—Increase in the health of the City of Dallas and its environment.

4.—Advantages to smaller towns and cities near Dallas, in helping them solve similar problems within their means.

* Irrig. Engr., U. S. Dept. of Agriculture, San Antonio, Tex.

5.—Encouragement of closer and more careful study and experimentation for further improvements along this line.

6.—Aiding the plan for making the Trinity River navigable.

7.—The psychology of the improvements on the people of other cities similarly situated.

8.—The development of the good-will of other cities on the Trinity River below Dallas.

9.—Facilitating extension of sewerage service to adjacent great industrial areas in the Trinity River bottoms, which areas cannot now be serviced except at a terrific cost to the city proper, or to individuals desiring to develop them.

10.—Encouraging development of parks, driveways, boulevards, swimming, fishing, and boating places, and the creation of additional lakes along the river in Dallas.

11.—No other available method of providing sanitary protection for adjacent communities which are unable or unwilling to finance necessary improvements. The old saying of Benjamin Franklin regarding hanging together or hanging separately applies forcefully.

12.—Permitting the district to build, say, 50 ft. of line for one connection, at a charge of, say, $25 per connection, will enable real estate men to develop worthy projects, while making it unnecessary for the district to hazard expensive systems for uncertain profits.

13.—Present system requires intercepters to be built at enormous cost to serve all possible development; under the sanitary district plan, each section would bear its proper share of the necessary development.

14.—Provision of a service charge to take care of necessary maintenance and operation.

15.—Assurance to owners of farm land adjacent to Greater Dallas that all future requirements will be met at once, when needed.

16.—Uniform instead of variable rates for adjacent communities.

17.—Constant employment to construction industries.

18.—Benefits to material men and contractors because of increased demand for their service.

19.—Avoidance of unnecessary duplication of work. In 1917, intercepters, pump, and disposal plant were built, which are now inadequate.

20.—A stimulus to sanitary organization in other communities, leading to co-operation.

21.—Advantage to State Board of Health, enabling it to handle similar questions in smaller towns.

22.—Appeal to business men in betterment of living conditions and prosperity of their customers and people.

23.—A logical continuation of the growth of sanitation. At first, proper sanitation in the home was the most important—then, sanitation of neighborhoods, communities, and cities proper. Now, proper sanitation of metropolitan areas is necessary for the public good.

24.—With the growth of paved roads and the education of the public as to the desirability of plenty of fresh air and sunshine, people are gradually

moving from the old congested areas of the cities to adjoining suburbs. These extensions are to sparsely settled districts, but to districts which may at some time become congested. It is unfair to expect the city proper to finance these long extensions and developments.

As to the last and best reason, Mr. Hemphill contended that sanitary districts permit the issuance of bonds to adequately financed collection and disposal systems. These bonds will be retired out of revenue as in the case of water departments.

GENERAL DISCUSSION

Widespread Benefit.—In commenting on Mr. Tanner's paper, C. E. Grunsky, Past-President, Am. Soc. C. E., stated that it illustrated very well the present-day tendency of making the charge for service proportional to the service rendered. He felt that, in this respect, engineers were frequently going too far.

"A sanitary district should really be formed for the purpose of providing proper collecting and disposal works and the detail of the bringing of the sewage itself to the collecting works might be left to the local communities."

Engineers must not forget "that every utility which is installed adds value to every piece of property in the community, and even those who do not get service should assist in paying not only for the installation, but also in some measure for operation." Mr. Grunsky felt that if the principle expressed by the author is carried too far, it may discourage some industries.

Fair Apportionment.—Samuel A. Greeley, M. Am. Soc. C. E., suggested that industries might possibly be separated in two classes. In one case he recalled this question was referred to an attorney of the District. This attorney,

"* * * went so far in his opinion as to say that not only was it the right and duty of the community to charge for the load, or service rendered, but if it did not do that it was discriminating against certain industries within its limits. * * * I have in mind one place where two industries have about the same assessed valuation with about the same payroll, each having a world-wide product to distribute, one with the ordinary domestic sewage as from 1 000 persons, the other with a domestic sewage equivalent to 200 000 or 300 000 people. Whether or not there is discrimination, there is a question of the industries and those interested to consider carefully."

Assistance from Industries.—According to James A. Childs, M. Am. Soc. C. E., who approved the content of the paper in general:

"If the cost can be apportioned in some fair and equitable manner among industries and domestic users of the system, it will encourage the industries to curtail or cut down to some extent the amount and strength of pollution which is added to the system."

Similar Problems in East Texas.—Mr. Thackwell declared that in Eastern Texas the practice generally is to charge the same amount for each connection. No attempt is made to compute the exact load and volume of waste from each source:

"As the growth of the community is increasing we find that industry is advancing and is putting a considerable load upon the disposal plants in the various towns in East Texas. For instance, the creamery waste at Marshall is a case. * * * We found that the washings from that plant were probably putting a very heavy load on the system and the charge to that plant was exactly the same as to anybody else in town. The laundry at Jacksonville has long been a source of nuisance. It has been discharging its waste directly into an open channel in the heart of town."

The method in which this problem was solved was to depend on the industries to co-operate for about one-half, and the city and community at large for the other half, of the load.

In financing sewage systems in various small towns in East Texas, it was felt that many people were unable to pay for individual connections. Accordingly, the city officials decided that it was better to charge a much higher rate to each individual user. Further, Mr. Thackwell stated,

"At Jacksonville, the laundry derives its power from a boiler and they use waste sawdust and the waste lumber that is at hand. Consequently, its power bill is very cheap. The owners have agreed to connect by belting up to a rotary fan and using a small activated sludge plant—they to pay for the plant and its operation, and we expect only partial purification. The city will take the large liquid load and handle it from that point on at the old charge."

Experiences Elsewhere.—In connection with the plans at Grand Rapids, Mich., the Engineers Club of that city, according to George M. Ames, Assoc. M. Am. Soc. C. E., recommended that the cost of the operation of the sewage disposal plant be assessed upon the users; that the city bonds be used to construct the sewer system; and thus that the plant be paid for as the general assessment. It was intended that the meter readings of the Department be used as a measure of the use of the sewers, and that a general charge of $3 per customer be made to cover the cost of office work. There was an additional charge of 3 cents per 100 cu. ft. for the service.

In approving the contents of Mr. Tanner's paper on general points, H. J. Hansen, M. Am. Soc. C. E., stated that,

"Mr. Tanner might have carried his analogy with reference to the public water supply just a little farther * * *. Whereas, the water supply protects public property from fire, the sewerage system protects the general public and city at large against disease and against nuisance."

Commenting on the question of disintegration of stone, A. T. Goldbeck, Assoc. M. Am. Soc. C. E., called attention to a so-called soundness test, in which rock is immersed in a solution of sodium sulfate for 20 hours. Then, after 40 hours, the process is repeated, and it has been proved that rocks will disintegrate as a result of that test. The action is mechanical rather than chemical, being due to the growth of crystals of sodium sulfate within the pores. In Mr. Greeley's opinion, this will no doubt serve to explain the disintegration of rock in some filters.

Calling attention to the disadvantages of assessing sewage costs on water-meter readings, Mr. Elrod, of Dallas, pointed to the fact that there were a number of private wells that supply water to various large buildings and some manufacturing plants in Dallas. The waste from this water must be carried by the sewer system, and yet no charge is made for it.

CITY PLANNING DIVISION

APRIL 25, 1929, 2:30 P. M., TO 4:45 P. M.

In opening the meeting of the City Planning Division, Chairman C. E. Grunsky called attention to the need of care in preparing papers for publication by the Society. After announcing the place of future meetings of the Division, and its aims, he introduced the first speaker.

CITY PLANNING PROGRAMS WITH SPECIAL REFERENCE TO THE LARGER CITIES

By Alexander F. Weisberg,* Esq.

Function of Consultant.—From the standpoint of experiences in perfecting the City Plan of Dallas, Tex., Mr. Weisberg deprecated the practice of turning the job over to an outside consultant, not because such an expert would be ineffective, but because each plan needs the group influence of the people whose actions the plan will affect. In his opinion,

"The consultant's plan is usually just a lifeless, physical plan—a pretty picture, to be sure, but little more than a pretty picture, unless you find a way to put life and practicality into it. This is not the fault of the consultant. His work simply has to be supplemented by others who can supply elements he is not so well equipped to supply."

Instead he recommended that the plan be prepared "by a very small, carefully selected working committee of competent, public-spirited citizens who are willing to devote real time to the task." There should be a competent consultant to work with the committee. During the formation period of the plan, the committee should hold numerous meetings, calling upon all city departments for data and an exchange of views.

A Workable Plan.—In short,

"The plan will be made by the community itself, instead of being imposed upon it by an outsider. * * * It must be big enough and broad enough to seize and hold the interest and imagination of the community, but not too big or ambitious for its financial resources. Thus, the plan should not stop with the city's probable requirements for streets, storm sewers, sanitary sewers, paving, sewage and garbage disposal, water supply and distribution, air port, fire department, transit facilities, and the like. It should also anticipate the city's probable requirements for schools, parks, playgrounds, boulevards, libraries, auditorium, community centers, Art Institute, and hospitals. * * * Wherever possible, the formulating committee should not be content with a mere statement of a general objective, but should make its studies and

* Chairman, City Planning Comm., Dallas, Tex.

its report as concrete and explicit as circumstances permit, by enumerating, where it can, the actual projects contemplated by the program, and giving an estimate of cost for each project. Thus, every citizen will be able to see and understand in a very definite way just how the program may benefit him personally, and the interest aroused by the program as a whole will be widespread and deeply rooted."

Various changes will be unavoidable, thought Mr. Weisberg, but nevertheless the program should be made definite enough to serve as a permanent guide and a mandate for the development of the city. One phase not to be overlooked is the legal aspects to be overcome. Another is the consideration of cost.

Financial Aspects.—If benefits of a project extend beyond the present generation of taxpayers, the costs should be likewise extended:

"The taxpayer of the future who will share in the benefits, should share in the cost. This does not mean that the entire cost should be deferred to future years. It may be regarded as sound financing to so arrange the schedule that a substantial proportion of the cost from year to year can be paid out of current revenue in the nature of a down-payment as the work is done, or, in part, on what is sometimes called the 'pay-as-you-go plan'."

"The entire program should be legally authorized at once", but the long-time obligation should be issued only as the work progresses. The probable growth of the city will influence the estimates of future taxable values and future revenue requirements. The taxes so fixed should be compared with other taxes in the city being planned and in other cities, so that in this way, quoting Mr. Weisberg,

"You can present to the taxpayer a comprehensive program which, over a period of years, will vastly broaden and improve all those economic, educational, and cultural facilities which a wholesome, well-organized and well-balanced community should have, and, at the same time, you can present to the taxpayer a perfectly definite, reliable, and comfortable plan of paying for these improvements without raising his taxes above the taxes commonly prevailing in cities of similar class."

Maintaining Ideals.—So much for the creation and adoption of the plan, which is not an easy task in itself, but the final task of putting the plan in operation, of protecting and preserving its integrity, is far more difficult. Thus,

"The first enthusiasm of the people too soon subsides as other things absorb their interest. Who can be counted on to stand upon the watch tower? And if the alarm be sounded, who will answer it?

"Obviously, the program, on its journey through the years, will find itself beset by dangerous and disintegrating forces. Against these, somehow, its integrity must be defended. It must be somebody's business to see that its essential plan, purpose, and rationale are not forgotten or distorted. Otherwise, the whole conception of planning the city's future fails, and we revert to the old haphazard, irregular, patch-work, time-serving, and wasteful expenditure of greatly augmented sums of public money. Under such conditions the voting of the program may well become a calamity."

Precautions for Dallas Plan.—The measures taken to insure the successful carrying out of the City Plan of Dallas may be listed under four main headings:

1.—Limitation of average annual expenditure by charter amendment.
2.—Charter amendment to force the governing body to secure in writing the advice of the City Plan Commission before beginning any new work.
3.—No bond money to be spent on any particular project until the general city plan for such character of projects has been prepared in advance by competent talent, with the advice of the City Plan Commission.
4.—Organization of a Committee on Supervision of Expenditures, having no official or political allegiance, to work in conjunction with the City Plan Committee.

Political Pitfalls.—Referring at some length to the natural difficulties and handicaps under which a city plan committee must necessarily work, Mr. Weisberg declared:

"Such a committee will encounter some officials who resent advice from any one except political bed-fellows of their own choosing. It will encounter other officials who will not believe that it desires merely to be friendly and helpful. It will encounter still other officials who will regard the very existence of the committee as an affront to them and its activities as an unwarranted intermeddling with their prerogatives. It will be charged with trying to run the city government and knowing nothing about it."

The dangers are most serious during times of political controversy, when, for instance, opponents may bring up issues that arose out of close decisions on the part of the committee. This has been the case in Dallas, Mr. Weisberg declared, but the mere existence of the Committee has no doubt been a real deterrent against evils that might otherwise have been encountered.

"*Public Approval.*—The ultimate value, however, of any advisory committee of this character is yet to be tested. Whether it shall finally be overcome by the influences which will always be operating against it is yet to be determined. Only one thing is certain. The value which the public will get out of it must depend upon the value which the public attaches to it. If a firm and wholesome public opinion enforces a *bona fide* use of the committee and protects it against political tampering with its personnel, it can perform a useful service. Otherwise, the kind of men who should compose such a committee are not likely to continue sacrificing their time without charge in a service the public does not desire. In this, as in all public business, the public will get what it really exacts. More it has no right to expect and does not deserve."

DISCUSSION

CO-ORDINATING COMMITTEE OF ENGINEERS

By A. P. Rollins,* M. Am. Soc. C. E.

Development of Dallas Plan.—The extent of the work authorized to date under the plan adopted by the City of Dallas, was outlined by Mr. Rollins, as follows:

1.—In December, 1927, the City of Dallas voted a bond issue of $23 900 000 for improvements recommended under the plan.

* Rollins & Clinger, Dallas, Tex.

2.—In April, 1928, the City and County of Dallas Levee District voted $6 500 000 for levees along the Trinity River.
3.—In April, 1928, the County of Dallas voted $6 950 000 for highways and boulevards around Dallas, and for new viaducts across the Trinity River.
4.—The City of Dallas voted additional funds for determining a relocation of the railroads so as to fit the plan for the ultimate development of the city.

A part of the work thus outlined, including the construction under passes for grade separation at railroad crossings, was recommended in the "Ulrickson plan" that preceded the present ones. The diversity of interest represented in the expenditure of these various funds made it seem essential that some co-ordinating agency be established. Accordingly, the Commissioners and Supervisors of the City and County and the Levee Improvement District decided to create what is now known as the Co-Ordinating Committee of Engineers.

Engineers Co-Operate.—Commenting on the accomplishments of this Committee, Mr. Rollins stated:

"So far, the Committee has been able to do much in the way of harmonizing the work under way, such as determining the proper location of viaducts, providing room for future track relocation, studying drainage problems, suggesting time for various phases of construction so there will be no unnecessary conflict on work, and in planning the program of work to be done. The governing bodies have welcomed all suggestions made by the Committee and have expressed an appreciation to the Committee for its recommendations.

"It is remarkable, in passing, that this Committee of Engineers, representing as it does different interests, has by frank and free discussion been able to iron out all differences of opinion and arrive at a unanimous one regarding all questions pertaining to public interest."

He commended the idea of such co-ordinating committees to other cities interested in planning problems.

OTHER DETAILS OF CITY PLANNING PROGRAMS

By W. J. Powell,* M. Am. Soc. C. E.

Execution of Plan Important.—Outlining briefly the various steps in the development of the Dallas City Plan, Mr. Powell represented the viewpoint of the City Plan Commission, of which he is Chairman.

The first concerted effort was in 1911. Many of the features of the present plan were also a part of the plan of twenty years ago; but, said Mr. Powell,

"Dallas had plenty of plan but little organization for its execution. Desultory progress was made from time to time, and one outstanding accomplishment is of record, that being the removal of the main line and switching facilities of a transcontinental railroad from the business district of the city and the conversion of its right of way into a broad thoroughfare, now one of the principal traffic arteries of the city."

The Metropolitan Development Association was organized in 1920. It was composed of nearly all the civil organizations of the city. A chief engi-

* Cons. Engr., Dallas, Tex.

neer was retained, under whose direction very complete zoning maps and ordinances were prepared. An adverse decision of the State Supreme Court made it impracticable to proceed with the zoning and further efforts were made under a newly created City Plan Commission, retaining the same chief engineer.

Final Success.—In 1925, a joint committee was appointed by the Mayor for the purpose of preparing a program of public improvements, including an estimate of the financial and legal machinery to make it possible. The result was "a most comprehensive report, covering all phases of civic development and setting up a schedule of public improvements to be executed over a period of nine years. The report also included a detailed analysis of the effect of the proposed expenditures on the tax rate for each year of the period." In due time, the city voted a bond issue of $23 900 000 to insure the successful completion of the present city plan.

The result has been an increased interest in planning by agencies outside the city. To this Mr. Powell ascribed the successful promotion of the project to reclaim land along the Trinity River and the bond issue authorized by the County for road improvements. These projects are more or less related to the Dallas program and together they form something in the nature of a regional plan.

He emphasized the fact that experience at Dallas has shown,

"The importance of thorough preparation and organization for the execution of a city plan as a part of the plan itself. The carrying out of a city plan is everybody's business, and unless controlled by a carefully prepared schedule of work based on a practicable financial program, it is not likely to go forward with satisfactory progress."

GENERAL DISCUSSION

A question by J. H. Johnston, M. Am. Soc. C. E., as to the details of the Texas zoning law, brought the explanation that a decision of the State Supreme Court, known as the Spann Act, was expected to prevent zoning in Texas. This Act has never been changed, but some recent decisions by the Supreme Court of the United States have made it seem certain that zoning can be carried out in Texas. Furthermore, there is an Enabling Act passed by the Texas Legislature, under which Dallas now has a new zoning ordinance which is about to go into effect.

CITY PLANNING AS RELATED TO THE SMALLER CITIES

By E. A. Wood,* M. Am. Soc. C. E.

A General Program.—The National Conference on City Planning has urged each American municipality to do three things:

* Cons. Engr., Dallas, Tex.

1.—Provide a City Plan Commission.
2.—Prepare and adopt a city plan.
3.—Secure public support for the administration of that plan.

The program thus outlined is general enough, according to Mr. Wood, to apply to municipalities of any size, location, or composition. The difference lies in the fact that, for the larger cities, any amount of corrective planning raises the relative cost. Therefore, to prepare a plan for a small town before it has grown extensively, is by far the most economical practice:

"If the official plan is prepared in advance of city growth, its enforcement becomes a matter of public support strengthened by the necessary rules and regulations to make it effective in all parts of the city."

Mr. Wood recommended the creation of a planning commission under conditions as outlined in "A Standard City Planning Enabling Act", issued by the U. S. Department of Commerce.

Personnel.—Specifically, he suggested that:

"The membership of the Planning Commission should be carefully selected and should be composed of outstanding citizens who can devote a considerable amount of time to the study of physical conditions on the ground as well as to the numerous meetings that will be required with different groups of individuals. The membership should be varied and if the plan is to be comprehensive and include all phases of city life, the public schools, park board, women's clubs, Chamber of Commerce, and retail merchants should be represented along with other organizations.

"Representatives from the City Government should be limited to a member of the Council and the City Engineer. The former should act as a liaison agent between the Council and the Planning Commission, and the latter is entitled to membership by virtue of his office. He is also charged with the execution of the plan and to some extent with the enforcement of the Commission's rules and regulations. The Mayor and City Attorney should be members *ex officio,* the Mayor by virtue of being the head of the Council, and the City Attorney by reason of his legal training."

Building Lines.—The creation of public interest is an important factor and this is easier in a smaller city because there is a narrower range of interest to co-ordinate. According to Mr. Wood:

"An early enactment of the platting rules is very desirable because of the failure of individual land owners to co-ordinate their subdivisions with existing streets, alleys, and blocks. This is not due so much to willful neglect as to the lack of foresight and the absence of a Master Plan to serve as a guide. Co-ordination of all plans is the outstanding feature of a Master Plan."

Zoning and building line restrictions are also important details that should be established in the necessary laws before the actual work of construction is begun. Quoting again,

"The establishment of building lines in residential districts adds materially to the privacy of homes, decreases the dust nuisance, decreases the noise from passing traffic, and encourages home beautification through the planting of grass, trees, flowers, and shrubs. They decrease the accident hazard to children by providing suitable playgrounds on every lot. They also permit vision clearance for traffic at street intersections."

Natural Development.—The plan should include ample provision for playgrounds and parks. Zoning regulations should be a part of the plan:

"In the early days of city life property classifies itself along economic lines and follows certain economic laws. Industries and warehouses are established along the railways, while the banks, hotels, and stores form a group around the public square. Beyond the stores are the homes of the people. Zoning follows these same laws and prohibits the intermingling of industry, commerce, and dwellings. Homes may be built in commercial or industrial districts, but commerce or industry cannot invade home districts, nor can industries invade business districts."

In conclusion, Mr. Wood stated that:

"Once the character of the various sections of the city is established by zoning, the street plan, the park, boulevard and playground system, the school locations, and the civic center can be designed and laid out as an integral part of the Master Plan which gives the picture of the future city. Of course, it will need to be expanded from time to time and even supplemented or changed. Nor will the work of the City Plan Commission ever be finished, but will continue from one year to the next, providing plans in advance of actual growth."

DISCUSSION

SPECIAL PROBLEMS IN SMALL CITIES

By J. L. Starkie,* M. Am. Soc. C. E.

The most essential difference between planning for larger cities and smaller ones was in extent, agreed Mr. Starkie. Fortunately for the science of city planning in America, municipalities do not progress very far without a prearranged plan before public consciousness is awakened, first to the need of corrective measures and then to the larger needs of improvements for future situations. This is a good influence because it does much to create the desired interest in the mind of the citizen.

Grade Crossings.—There are some planning problems peculiar to smaller cities. For instance, thought Mr. Starkie,

"There is more of a tendency to impose upon the engineer for his incorporation in the plan, ideas allegedly advanced for the alleviation of existing faults, but with no outlook on the future. Many merchants in our smaller cities regard a traffic congestion as a sure sign of progress and to be preferred. Fortunately, however, our Texas highway engineers independently of city planners, or co-operating with them (almost universally against strong pressure), are insisting upon highway locations from a broader viewpoint so as to by-pass through traffic around congested population centers, and avoid railroad grade crossings wherever possible."

The elimination of railroad interference with traffic on thoroughfares is an important part of city planning programs. As compared with the number of grade crossings eliminated, the increase in the number of new grade crossings is alarming:

"So the City Plan Engineer who ties into every railroad grade-crossing separation, which he proposes, the closing of one or more existing grade crossings of the railroad, serve humanity in general as well as the city which employs him."

* Brownwood, Tex.

A Gradual Process.—The idea of a city plan should not be to tear down at once all existing structures that appear undesirable, but to create such a picture of the whole situation that changes can be made from time to time as the proper occasion presents itself:

"A city plan once adopted should remain in operation perpetually (subject to such modifications as changing conditions demand) and the accomplishment of its objectives made the matter of a gradual process with its complete realization possibly never fully attained. Our thrifty railroads are continually changing their standards of various physical properties, but the new facilities are, except in the rarest of instances, not installed until the old ones have completed their useful service life or unusual economic conditions warrant a change."

CITY PLANNING IN SMALL CITIES

By O. H. Koch,* M. Am. Soc. C. E.

A small city, according to Mr. Koch's definition, is one having a population of 50 000 or less, which is the type that predominates in Texas. Another study of differences in planning viewpoints could be made for cities between 100 000 and 200 000 inhabitants and great metropolitan centers.

Predicting Needs.—In a small city the first step is to take an inventory of existing conditions. The next step is to predict the future needs by studying the past record of growth. Mr. Koch especially believes,

"That city planning should be at least 90% preventive and 10% corrective. The accuracy with which this prediction is made will determine, to a great extent, the ultimate value of that city plan.

"It is relatively easier to make such predictions in the large cities on account of having available more definite records and tendencies of growth upon which to base predictions. In the small cities this particular information is somewhat limited and we have, as it were, a shorter base line from which to project and predict as to the future tendencies. In the large, more established cities, the trend of growth and type of development is more firmly established and its general character more permanently fixed."

For instance, a small city may be supported primarily by educational institutions. A very satisfactory plan may be developed on the basis of these conditions only to be entirely overthrown, if in subsequent years a large industry moves into the town. Under such circumstances, a small city would have to decide if it were worth while to plan for such contingencies in the future. A large city might not be so seriously affected by the addition of a single large industry.

Handicaps of Small Size.—Relative costs are a serious problem for the smaller cities. As Mr. Koch remarked,

"If the planner is skeptical in regard to the city's growth and tries to be too conservative in planning for the immediate future only, in order to hold down cost, he will not be securing the full measure of benefit to be derived from city planning. On the other hand, if he is too optimistic and plans for

* Dallas, Tex.

too many years in advance, he is apt to cause exceedingly large expenditures for immediate use and to have excess of expense and unused facilities.

"The excess cost of the superfluous facilities, the overhead carrying charges, and the maintenance of the same capitalized over a number of years would very readily exceed what the cost would be with a more conservative plan, even if the additional corrective measures had to be taken after a period of years. It is this choosing of the happy combinations that will determine the comparative value which any city plan may develop."

Provisions for parks and other recreational facilities can be made much more flexible in the larger cities, because of the larger number to be supplied. In the smaller cities, on the other hand, the probable first cost per capita will be an effective deterrent against planning too far in the future.

Various Difficulties.—Zoning for business, residential, and industrial sections presents especial problems in a small city, because there is not so good a possibility of providing transitional property that can be changed to permit later changes in zoning status.

Small communities are not justified in spending money to eliminate objectionable grade crossings as in the case of larger cities which plan this feature as a major operation. In general, Mr. Koch states,

"City planning in the beginning was born of necessity due to congestion and intolerable conditions in the larger cities. Most of the pioneer work in this field was developed in order to relieve such conditions. It is these obvious advantages and the favorable results which have been made possible through city planning. * * * The relief gained from these intolerable and acute conditions has encouraged the city planning principles to be applied to the preventive phase, and has educated the public to the realization of the great advantage of planning in advance. This work * * * has been most intensive in the larger cities and the methods and principles adopted were designed to suit conditions in such cities. With the advantages thus favorably established the move naturally is spreading into other cities as well."

GENERAL DISCUSSION

Standardization of Map Symbols.—It is highly desirable to develop a method of standardizing the symbols used on city planning maps for zoning purposes. For instance, according to Mr. Starkie, on a certain railroad line there are a number of cities adopting city plans, but the engineers in each city are using different symbols to indicate the same thing. Mr. Wood replied that methods of standardization are now being developed in California, and are also being gradually extended into other sections of the country.

Morris Knowles, M. Am. Soc. C. E., called attention to one plan,* which, although it has not been approved, contains many suggestions that might be useful for smaller communities. It contains methods of standardizing heights of buildings, designation of occupancy in lots, etc. Mr. Knowles concurred in the opinion of Mr. Weisberg especially that the city plan:

"Which lives is the plan made by the people themselves, the representatives of the taxpayers in a town. It becomes more naturally the production

* *City Planning Quarterly*, January, 1929.

of the home community and is not, as Mr. Weisberg says, handed to the city fathers and put in the vault. It is more likely to live because it has become the instrument and work of local people."

Opportunity of a Small City.—Successful city planning involves three steps: First, the inception, that is, the propaganda and the arousing of interest; second, the creation of the plan itself; and, third, the construction work involved in bringing the plan to fruition. Mr. Knowles especially called attention to the fact that communities of from 2 000 or 3 000 to 25 000 people have a much simpler problem than the larger cities, because less of the planning is re-planning, and, consequently, it should be relatively less expensive.

"As I conceive it", Mr. Knowles declared, "the future of planning is in the small communities of the country, which have still to grow and still to come to their full size, because they can avoid the mistakes of the past."

In any case, planning and even re-planning cannot be adjudged as costly as the method of letting the city grow as it will, because under the latter system, continued changes, such as grade crossings, improvements to prevent accidents at narrow street intersections, etc., will in the end require far more money than the total cost of creating a Master Plan with its subsequent natural development.

Chairman Grunsky called attention to the number of sky-scrapers that are permitted along narrow streets, where it is almost a foregone conclusion that new problems of congestion will be the result. He noted that a closer co-operation with the legal profession seems to be indicated by the discussion. One subject worthy of consideration would be to require that builders design their structures so that sidewalks can be built in arcades back of the building lines as the need for street widening develops. According to Chairman Grunsky, they should learn by the mistakes made elsewhere, and so tax the zone above the tenth story that no one would desire to build higher than that.

Municipalities in the Chicago Region.—City planning in the metropolitan districts involves much of designing for smaller cities. The Chicago Region, for instance, extends 150 miles from Chicago, and, according to W. W. DeBerard, M. Am. Soc. C. E., embraces 3 States, including 13 counties and 1 000 municipalities. The complexities involved in the relation between these smaller entities and the larger cities have been brought about by modern transportation and communication facilities. The relation between the two systems, as far as city planning is concerned, must be co-ordinated.

Commenting on the Committee of Co-Ordinated Engineers in Dallas, Mr. Weisberg closed the discussion on his paper by declaring that its work was a valuable contribution to the general problem, that is, protecting the integrity of the entire plan.

"I think", he stated, "it would be very helpful if there could be established, and no doubt there could be, a proper co-ordination between that group of engineers and the other planning bodies of the community so that the other laymen planning bodies of the community may have the benefit of the judgment of this group of engineers who are in direct touch with all these projects of these several corporate bodies."

IRRIGATION DIVISION

APRIL 25, 1929, 2:30 P. M., to 5:00 P. M.

The author of the first paper on this program being unavoidably absent, Chairman J. B. Lippincott, designated F. C. Scobey, M. Am. Soc. C. E., to read the prepared manuscript.

IRRIGATION IN MEXICO

By Sr. Ignacio Lopez Bancalari*

In an outline of the activities of the Mexican Government in preparing to meet the irrigation problems of that Republic, Sr. Bancalari described first the topography of the country as it affects rainfall characteristics. He rounded out his paper by a detailed description of five large projects.

Topography Affecting River Flow.—Mexican topography, according to Sr. Bancalari, may be conceived in the form of a rough pyramid, with a triangular base, and with vertex in the great mountainous part of the State of Oaxaca. The northern face of the pyramid has a relatively moderate slant toward the United States border, while the two sides toward the Pacific Ocean and the Gulf of Mexico, respectively, have grades that are comparatively steep. The result as it effects the action of Mexican rivers, is to make the flow generally torrential in nature.

The character of most of the rivers through the alluvial soils in the lowlands is marked by a scarcity of flow during low rainfall, abundant flow during the rainy season, and, in a few regions, an occasional case of concentrated flow due to melting of snow. In brief, the rivers are swift because of the steep grades in the upper sections. In the alluvial lowland valleys, the rate of flow moderates considerably, and the rivers widen until they finally reach the various deltas or estuaries where they follow the general law of fluids meeting a great mass of water, such as the ocean. Some of the smaller streams end in lagoons from which the water is slowly evaporated, or from which it filters away as ground-water, turning the area into salt marshes which are useless for agricultural purposes.

As Sr. Bancalari explained, the irregularity of flow, especially the torrential flow of rivers during rainy seasons, makes it necessary as a fundamental principle to provide storage reservoirs and other regulatory devices for a uniform distribution and control of the water. From this point of view, Mexican engineers, during the past five years, have given special attention to

* Director, Comision Nacional de Irrigacion, City of Mexico, Mexico.

the flow characteristics of the Hazas River. In two of these years the flow was scarce; in two, it was plentiful; and, in one, it had a markedly acute maximum and minimum during the year.

Other streams seem to be subject more or less to the same periodical flow characteristics, and it is this that makes stream regulation an indispensable part of any program of hydraulic importance in Mexico.

State Policy.—According to Sr. Bancalari, the policy of the Mexican Government since 1910 has been to establish large public works of this nature under conditions such that the money expended would serve the greatest good to a large number of relatively small land holders, rather than one large land holder. Private land owners have been unwilling to invest the large sums necessary for works of this kind without receiving the protection of the Government on the capital thus invested, and the need of immediate financial returns to the individual owner under those conditions has tended to create large land holdings by a few, to the detriment of many small land holders.

As a result, according to Sr. Bancalari,

"Such conditions made the Mexican State realize that works destined for the most convenient agricultural use of water from our rivers, should not be left to the direction or patrimony of capitalists or of private irrigation enterprises, but that their control and operation should be regarded openly as a public function performed by the State, so they could have all protection and care, not moved by selfish motives, but by the lofty and patriotic purpose of helping the great majority of people by the investment of public funds."

In other words, the Mexican Government does not look with favor on the tendency of large interests to consolidate. It remains firm in the belief that the greatest good comes from promoting the feeling among a large group of land holders, in order that they have a mutual and personal interest in the development of the land on which they work. Sr. Bancalari referred to the present policy of his Government as,

"A disinterested and generous program, impersonal and highly patriotic, to give to our rural class the comforts that were totally unknown to them during the somewhat feudal régime inherited from the Spanish colony and which was still in existence, with slight modifications or none whatever, up to the social 'commotion' of 1910."

The National Irrigation Law under which present activities are being developed, was enacted in 1926, and in this very short period of time, five large projects have been practically completed, opening to the benefits of irrigation an area of about 600 000 acres (250 hectares). Undoubtedly, many errors have been committed, but it is also evident that the Government has profited by them, and that there will be greater improvement in future work of this kind.

The work performed by the State on irrigation projects could have been simplified if it had been confined to Federal property, or if the Mexican Government had been able to afford the luxury of acquiring the property necessary for installing irrigation works before beginning any project. Unfortunately, according to this authority, neither of these practices was feasible, because, in the first case, the ownership of most of the National lands had been trans-

ferred to private citizens, and the land that remains is not very thickly populated; and, in the second case, because the money involved would have made such project prohibitive. This is the reason why the Mexican Government has directed its attention to the cultivation of privately owned lands under special conditions imposed by law.

Compulsory Irrigation.—The National Law of Irrigation established as a general principle the condition that the owner is under obligation to irrigate his land, and for this purpose he is furthermore under obligation to use water provided by the Federal Government wherever possible. If he cannot or will not accept and fulfill this obligation, then the Government will irrigate the land and require the owner to pay his share of the cost by forfeiting a part of his land equivalent in value to the total cost of the work performed. This land value is fixed by considering the hectare cost of irrigating the project of which the farm is a part.

Since the law has been enforced, the National Government has been very active applying it to provide irrigation where needed, "and the land owners affected thereby have expressed in most cases their willingness to comply." Hence, irrigation in Mexico is not considered so much in the nature of a right as it is a duty imposed on the land owner.

There have been five projects inaugurated since 1926 that are now completed, or nearly so. These are the so-called President Calles System, the Rio Mante System, the Rio Salado System, the First Irrigation System of the Lerma River, and the Mezquital System.

President Calles System

This system, as Sr. Bancalari explained, was one of the first projects undertaken by the National Irrigation Commission, and was named in honor of the then President of Mexico. It is located in the State of Aguascalientes at a point between the City of Aguascalientes and the railroad station, La Punta. It extends northward to San Pedro Piedra Gorda, in the State of Zacatecas. The waters that irrigate the State are collected from two main drainage basins. The North Basin drains into the San Pedro River through the Pabellón, Santiago, and Chicalote Rivers. The West Basin drains into the Calvillo River, through the Tejas and La Labor Rivers. The topography in the State of Aguascalientes is in general very broken and rugged. The area irrigated lies at an average elevation of 6 170 to 6 350 ft. above sea level. The climate is typical of that in the upper valleys in the Central Plateau, and may be classified as temperate. The temperature is subject to moderate changes. In general, the region is healthy, and favorable conditions prevail.

The rainfall records taken in the City of Aguascalientes for a period of thirty-six years show considerable changes. During the dryest year the minimum rainfall was 3.27 in., and in the year during which the precipitation was greatest, the rainfall was 33.47 in. Almost 80% of the annual rainfall occurs in the period from June to September. The spring showers represent 5.3% of the yearly rainfall, and the winter rains, 6 per cent. In winter, however, the precipitation is very irregular, being great in some years and very slight in others. Within the area affected by the project, there are no less than 33 communities with a combined population of 9 000 inhabitants.

Available Water Supply.—The reservoir to be formed by the dam on this project will have a capacity of 79 000 acre-ft. The reservoir basin of the Santiago River has a total area of 97½ sq. miles. After a number of studies, it was decided to compute the average run-off as 35% when the rainfall during any month is more than ¾ in. Studies were made to determine the suitable storage capacity necessary to regulate the flow of the river, based on its yearly discharge rate. Thus, it was determined that the storage should be 84 000 acre-ft.

Likewise, numerous investigations were made to determine probable evaporation losses in the reservoir, stated Sr. Bancalari. It will not be possible to determine canal losses until after the system has been in operation, but a comparison with other projects similar to the one under study, led to the conviction that the loss would probably be 40 per cent. In studying the water requirements for growing crops, it was estimated that about 47 000 acres of land could rely on the run-off from the Santiago River System for irrigation purposes. The water from the Pabellón River will serve as a supplementary supply to the President Calles Project.

As in the case of the Santiago River, there are no exact rainfall data available for the Pabellón River, but it has been determined that its total flow can be diverted into the main reservoir by means of a tunnel having a capacity of about 530 cu. ft. per sec. Without the supplementary supply from the Pabellón, water stored in the reservoir could be depended upon to irrigate only about 34 000 acres.

Soil Studies.—The soil in this project has been classified under five different headings. The first (Class 1), is of the better quality, located along the edge of the river. It is free from alkali, its underground drainage is good, the topography is such as to make it suitable for irrigation, and its composition makes it a good soil for various crops, including fruits and vegetables.

Class 2 has the same characteristics as Class 1, except that the stratum is generally not so deep, being in no case more than 1½ ft. Class 3 is an inferior soil compared with Classes 1 and 2, the points of inferiority being principally in relative drainage facilities. Soils in Class 4 are of a rocky nature, which is difficult to plow, and which would require expensive drainage work; and in Class 5 are included the soils that are absolutely unsuited for irrigation purposes, either because of the topography or their chemical composition.

The Dam and Its Outlet.—The site selected was such as to provide for the necessary reservoir capacity with an arch dam 187 ft. high. At the top, the dam is about 920 ft. long, composed of about 333 ft. along the center arch, 193 ft. of gravity spillway section, and about 394 ft. in two straight gravity sections. The total height above the foundation is about 220 ft., and the dam contains about 60 000 cu. yd. of concrete. The discharge capacity is about 250 000 cu. ft. per sec., or about four times the maximum rate of flow observed in the river. Four pipe lines leading from the reservoir, are designed to supply the irrigation and power needs of the region.

The two irrigation pipe lines are 54 in. in diameter, and two for hydraulic power are 66 in. in diameter. All the valves are located in chambers at the foot of the dam. In addition to the detailed description of the dam, Sr. Ban-

calari also explained the construction features of the irrigation canal, including a tunnel which is cut from the reservoir through an adjacent hill. This tunnel, 11½ ft. in diameter, was drilled through solid rock for a distance of about 4 300 ft. The grade is 0.2 per cent. From the end of the tunnel north to the San Pedro River, and then south to the Chicalote River, a distance of about 17½ miles, the canal is 20½ ft. wide at the base, with battered slopes.

The construction of the canal involved special problems dealing with siphons for passing water under the more important rivers and streams, aqueducts for crossing over small streams, dikes and outlets for feeding branch canals, spillways for the control of excess water, highway bridges over the canal, and special bridges at railroad crossings.

Secondary Canal.—In planning the distribution system, the Irrigation Commission took great pains to insure the most convenient and profitable use of the land as an essential feature of this project. Each parcel is to be served by an individual canal which is to be readily accessible from a public road. The maximum area of land allotted to any farmer will be about 370 acres, although the average size of the farms is more nearly 120 acres. Up to the present time the cost of the work has been $7 412 000, which is equivalent to about $158 per acre. However, in order that the operation of the project may be carried out on a favorable basis, the Government has arbitrarily fixed a price of $260 per hectare, which is equivalent to about $105 per acre.

The "Rio Mante" System

Unique Characteristics.—As explained by Sr. Bancalari, the problems involved in the construction of this project are exceptional. The Rio Mante originates in a spring in the Cucharas Mountain. The spring has an erratic flow, the minimum being as low as 280 cu. ft. per sec., and the maximum, 3 180 cu. ft. per sec. The flow of this stream can be carried directly to the lands that are to be irrigated so that storage will be unnecessary. Therefore, the purpose of this project is merely to secure a satisfactory diversion system, and the construction of the units necessary for the best utilization of the water.

The lands subject to irrigation have an area of about 45 000 acres. The average elevation is about 165 ft. above sea level. The maximum spring flow of the Mante River occurs near the end of September, and the minimum near the end of May. At a distance of about 9½ miles from its source the river is divided into three main branches that form its delta. These branches cover an area of approximately 9 400 acres which are flooded by the river and are, therefore, of little value for cultivation.

Drainage.—Instead of irrigation, this area requires a complicated drainage system so that some of the land may be reclaimed for agriculture, and so that all of it may be made healthful, since the region is a notable malarial district. Everything considered, however, Sr. Bancalari declared that this project has been the simplest of all those undertaken thus far, and the lands have a high agricultural value.

Underlying the surface soil of this region is an extensive stratum of impermeable slate, which lies at distances varying from about 1½ to 6½ ft., and tends

to drive the water toward the surface of the ground, resulting in the formation of swamps. The structures involved consist only of a diversion dam, with the flood-gates at the heads of two main canals. The diversion dam is merely a common type of dike 31 ft. long and high enough to raise the water level from about 3¼ to 10 ft. The dike itself is 13 ft. high, and the water flowing in the river is about 2.6 ft. deep.

Completion.—The land to be irrigated is a mixture of clay, silica, and humus. It is a very rich soil, and the climate is warm, so that the region is especially well suited for the cultivation of sugar cane. The land is owned by a progressive class of farmers who are anxious to use and adopt the most modern agricultural methods. For this reason, even if the project is small, it must be regarded as one of the most important.

In addition to the structures already described, this project includes the building of decanting and settling basins, gates for the distribution of water, siphons, aqueducts, bridges, and railways. Provision is made for a complete network of secondary canals for irrigation, and a complementary system of pipe lines for drainage. This project is now completed and ready to be placed in operation at a cost of $4 109 500, or $228.30 per hectare, which is equivalent to about $92.50 per acre.

The "Rio Salado System"

The Project.—This is the largest of the projects now under construction, stated Sr. Bancalari. It is particularly important because the present Government in Mexico depends on it especially for colonization. Due to the irregular occurrence of rains, it has heretofore been very difficult to utilize the land for agricultural purposes, and the area is practically uninhabited.

It is hoped that with the help of irrigation, this region will become one of the most active centers of population for agricultural production. It contains approximately 740 000 acres of land. It is expected that all the water from the Rio Salado will be used for irrigating purposes. This river has its source in the northern part of the State of Coahuila, passes through the State of Nuevo León and terminates in the Rio Bravo, in the State of Tamaulipas.

Structures.—The main dam has a length of about 3 950 ft., and the embankment is about 3 300 ft. long, built of earth taken from a near-by pit. The maximum height of the embankment is 111.5 ft. The thickness at the crown is about 20 ft. The up-stream slope, 1 on 1¾, is paved with reinforced concrete. The down-stream slope is 1 on 2. In order to provide the storage capacity required in the reservoir, it was necessary to close a series of ravines by constructing about 5.8 miles of earth embankment with an average height of about 30 ft. The total capacity of the reservoir thus formed is 1 100 000 acre-ft. With this quantity of water available, it is expected that about 160 000 acres of land can be irrigated.

The irrigated canal leading from the extreme north end of the dam has a discharge capacity of about 2 300 cu. ft. per sec. In the clear its breadth is 59 ft. The total length of the finished canal will be about 113 miles. The section of the canal diminishes for the lower 85 miles, or so, to a capacity of about 900 ft. per sec. The lands that will be irrigated are of a high grade for

agricultural purposes, and will be especially good for cotton, alfalfa, and other crops that grow in a high and dry climate.

A survey has been made of about 495 000 acres, with the result that some 160 000 acres have been chosen for irrigation, leaving the remainder, which is of a relatively inferior quality of land, for pastural purposes. The estimated cost of this project will be $35 908 000, or $552.43 per hectare, which is about $225 per acre.

First System of the River Lerma

General Conditions.—This project is being developed for the double purpose of irrigation and manufacture of hydro-electric power. The latter has been leased to a private company. Sr. Bancalari explained that the River Lerma has its source in Lerma Lake, in the State of Mexico. It receives its supply from rainfall and from the melting snows of the Toluca Mountain, as well as the high hills that surround the lake. It runs through the States of Mexico and Guanajuato, through the Lake of Chapala, from which it emerges again as the Rio Grande de Santiago. This river finally empties into the Pacific Ocean after traveling a total of almost 375 miles across the most thickly populated and best agricultural regions of the Republic. This region has been cultivated so intensively since early Colonial days, that the upper stratum of soil is fairly well impoverished, so to speak, but it is hoped that by means of modern irrigation methods the benefit can be taken of the richer soil below the surface.

The storage dam has been constructed at the little town of Tepuxtepec, in the State of Michoacan. Its length is about 1 970 ft., and its total height above the river bed is about 150 ft. Storage material at this site was so poor that an additional excavation of about 115 ft. was necessary before any reliable foundation rock was reached. Even at this point it was necessary to fill various seams and fissures by pressure grouting to a further depth of from 160 to 200 ft. It was necessary to build a diversion tunnel about 920 ft. long, around the dam during construction, and this tunnel will be used for flood-control purposes.

Power Development.—On account of unsatisfactory conditions obtaining at the site, stated Sr. Bancalari, special efforts were made to obtain impermeability. A hydro-electric plant below the dam will receive about 1 660 cu. ft. per sec., under a total head of 607 ft. The tunnel leading from the storage basin to the hydro-electric plant is 13.8 ft. in diameter, and about 2 miles long. At the end of the tunnel there will be special penstocks to conduct the water to the plant.

While the hydro-electric rights have been granted as a concession to private individuals, the ownership of the storage works and water will be retained by the Government. The Government maintains the right to stipulate the fixed service plan under which the private company will develop hydro-electric energy. The storage basin is planned for a capacity of 405 000 acre-ft., and the quantity of water available for irrigation and water-power purposes is expected to be 600 cu. ft. per sec.

The Valle de Mezquital System

Water Sources.—This project is unique from the point of view of the very small sub-divisions into which the land is separated. The run-off on this drainage area flows in two main river systems; one of these is the Rio de Tula, which becomes, successively, Rio Moctezuma, and Rio Panuco, before it discharges into the Gulf of Mexico at the Port of Tampico. The flow of this river system is controlled by two reservoirs with a total capacity of 65 000 acre-ft.

There is another source of water from the Tequixquiac Tunnel, from the drainage system in the Valley of Mexico, which empties into the Rio Salado. The rate of flow is from 106 to 530 cu. ft. per sec. This latter supply is utilized at various places for hydro-electric energy as well as irrigation. Together, these two main sources of supply are sufficient to yield water for about 54 000 acres of very good land, subdivided to such an extent that some of the owners have no more than from 1 to 1½ acres.

Water Uses.—Quoting Sr. Bancalari,

"The waters from the drainage of the Valley of Mexico emerge from the Tunnel of Tequixquiac and reach the Rio Salado, from which they are diverted by means of a small dam, constructed in Tlamaco, to irrigate the southeastern section of the System; from there they go through the power plant of 'Juando', where approximately 1 200 kw. are produced, to a second canal which carries them to the next plant, 'Canada', with a capacity of 800 kw.; and, finally, from this plant, either they flow into the irrigation system of the lowlands in the Valleys of Tepatepec and Actopan, or they continue through a new power canal up to the 'Elba' plant (at present shut down on account of irrigation necessities), from which they return definitely to the Rio Tula."

The first of these systems was begun by private enterprise for the purpose of forming an irrigation company to benefit the lands belonging to the individuals comprising that company. The activity of the Government, however, in developing other irrigation projects in this neighborhood caused the private owners to seek the Government's aid in solving their own problems, with the result that the Government finally acquired the private system and added it as a part of the National program.

The total cost of the Valle del Mezquital works to date is $1 345 600, or $61.16 per hectare, which is equivalent to about $25 per acre. Operating expenses of the system are paid by revenue from operation, and it has shown an annual profit of $30 500 which corresponds to 2¼% interest on the capital invested.

Additional Projects

In his paper Sr. Bancalari confined himself largely to works that are completed, or are so nearly completed that their construction problems have mostly been recognized and solved. Besides the projects mentioned, ten other large projects are being studied. Special commissions have finished final plans for work on (1) the Rio de Tijuana, in the northern part of Lower California; (2) Rio Yaqui, in the State of Sonora; (3) Rio Conchos, in the State of Chihuahua; (4) Rio San Juan, in the State of Nuevo Leon; (5) the project on the Arroyo de Santa Gertrudis, in the State of Tamaulipas; and (6) the projects on the Rivers Balsas, Nazas, Lerma, Grande de Morelia, Aguanaval,

etc. It is expected that before 1935, these works will provide for the irrigation of not less than 4 900 000 acres, at the same time creating employment through the development of a hydro-electric industry capable of generating not less than 400 000 kw.

All this work may be classified under the head of progress, and need not be further described. Sr. Bancalari acknowledged the indebtedness of the Mexican Government to the co-operation and the very valuable technical help of the J. G. White Engineering Corporation. With this help, and encouragement, there is being developed an enthusiastic group of native Mexicans who are becoming experts in the various branches of irrigation design and construction. He paid tribute to the valuable influence of former President Calles, and to President Emilio Portes Gil.

The Mexican National Irrigation Commission in the City of Mexico is a remarkably efficient one, according to observations of Chairman Lippincott during a recent visit. As compared with work done in the United States, progress is much slower because the Administration prefers to train native Mexicans. The result, however, is quite satisfactory and agreeable.

DISCUSSION

THE MEXICAN POLICY

By Armando Santacruz, Jr.,* Jun. Am. Soc. C. E.

The National irrigation laws of Mexico, stated Sr. Santacruz, were enacted three years ago (1926). Since then, more than $5 000 000 per year has been expended on irrigation work. In Central Mexico, where the first developments were inaugurated, the Government has planned the division of these lands into parcels, ranging in size from 60 to 300 acres. According to Sr. Santacruz:

"The land is open to anybody, Mexican or foreign, and they have twenty years in which to pay for it, without interest, and the Government will establish land banks to finance the settlers."

Settlement.—Efforts are being made to profit by the mistakes of others, but allowance must be made always for differences in social and economic conditions between regions set up for comparison. Mexican land near the Lower Rio Grande is just as good as that in Texas across the river, but marketing conditions are different. One very important difference is that the Mexican farmer is limited in his knowledge of agriculture.

"The present plan of the Government", Sr. Santacruz stated, "is to bring back into Mexico the laborers who are now in the United States. Most of them have a good knowledge of agriculture, which they have learned, and they know

* Cons. Engr., Mexican Section, International Boundary Comm., El Paso, Tex.

how to handle tools and machinery, and many are glad to go back to Mexico to get a piece of land and settle on it. The Government gives preference to these people, and is looking forward to development along the border."

The Mexican policy differs from that established by the National Reclamation Law of the United States, in that when certain percentages of owners desire irrigation development, they can request the Government to build for them. When this is done, the Government collects the money in twenty years, in cash or in land. Some of the big land owners in Mexico have millions of acres, and many of them are willing to exchange from 25 to 40 or 50% of their land, in order to have the advantages of irrigation on the remainder.

ACCEPTANCE OF LAND AS PAYMENT

The advisability of accepting land as payment for expenditures made on irrigation projects is of rather doubtful value, according to W. L. Drager, M. Am. Soc. C. E., who is working on the Calles Project. This project is now about 90% completed, at a cost of about 4 000 pesos per hectare. The value of the land to be accepted as payment is now about 10 pesos per hectare. He emphasized that,

"The cost the Government has set up on this land is 260 pesos per hectare, so you see it would take more than the entire area of the land of the farmer to pay the irrigation cost of the project."

SEEPAGE PREVENTION AND DRAINAGE IN THE EL JARDIN DISTRICT

By Ernest L. Myers,* M. Am. Soc. C. E.

An Unusual Site.—The El Jardin District is located east of Brownsville, Tex., within a large loop formed by the Rio Grande. It is the most easterly district on the Lower Rio Grande, and between it and the coast there are mostly salt marshes and shallow lakes, having elevations "but a few feet above the sea level". Occasional ridges of higher land rise to 20 or 30 ft. above the general level.

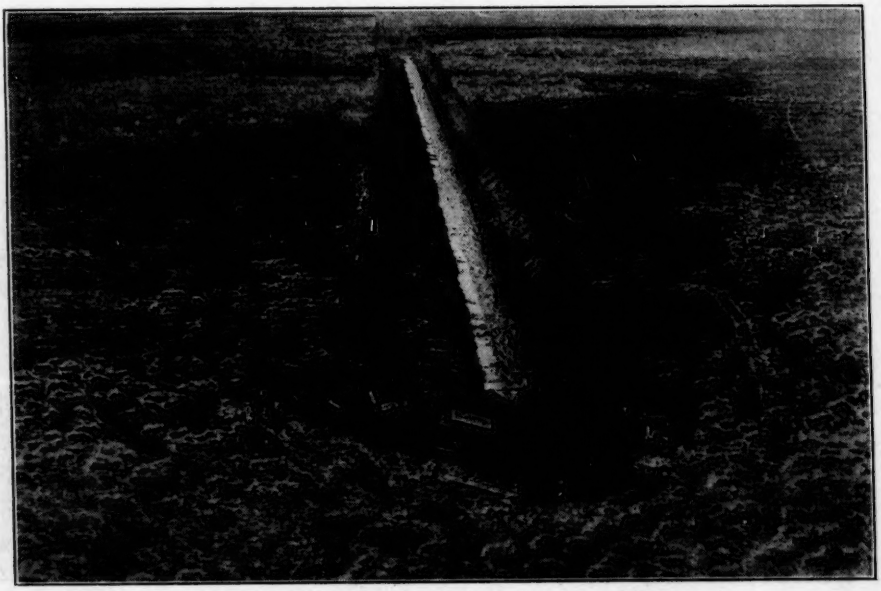
This district is within the coastal plain that extends back from the Gulf of Mexico. The soil is composed of quite salty black clay. Mr. Myers explained that,

"The Rio Grande River has cut across this plain in long loops, forming channels in the natural soil and building up banks of light-colored sandy silt. It has from time to time cut off loops and added new ones, eventually abandoning the old channels entirely. Some of the abandoned channels have entirely filled, leaving long, low winding ridges 10 or 15 ft. above the natural level of the plain. Others are now long narrow lakes, known as 'resacas'".

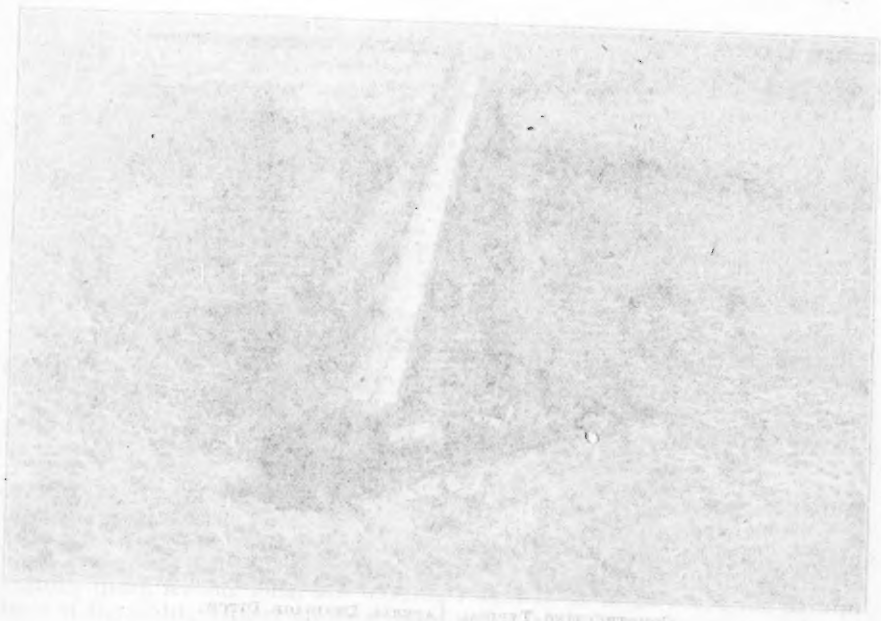
The underlying soil varies from a light-colored sandy silt to a black clay. In many localities, the land remains in approximately its original condition

* Cons. Engr. (Myers & Noyes), Dallas, Tex.

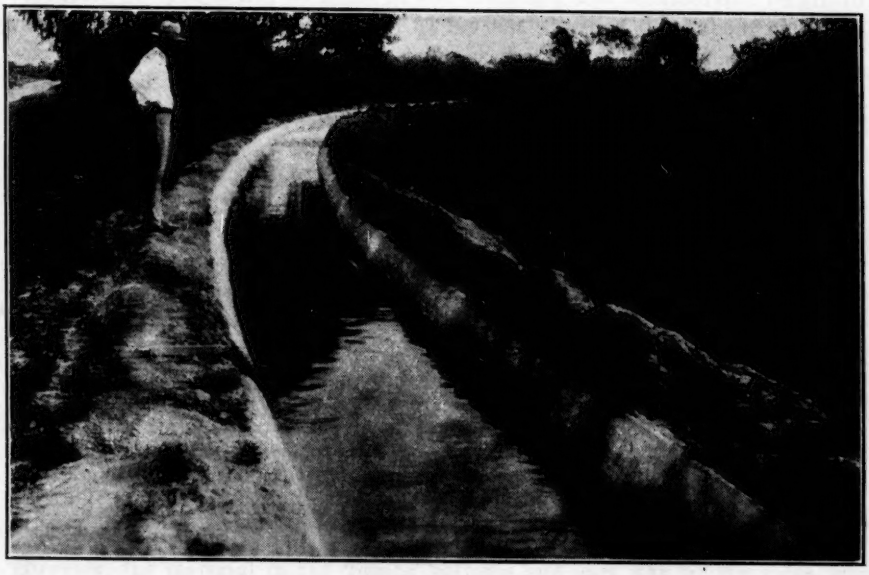
BIRD'S-EYE VIEW OF EL JARDIN DISTRICT.



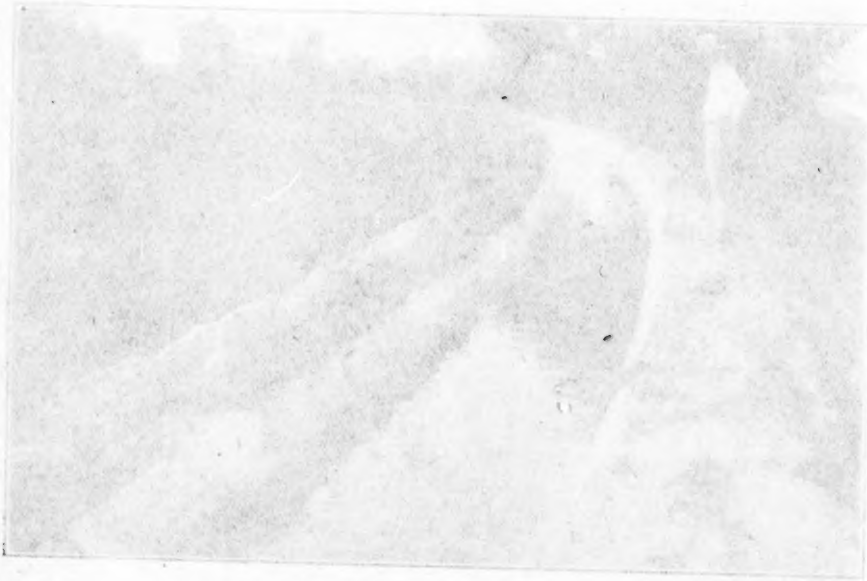
CONSTRUCTING TYPICAL LATERAL DRAINAGE DITCH.



EXCAVATING DRAINAGE DITCH THROUGH "RESACA."



CONCRETE-LINED CANAL, EL JARDIN DISTRICT.



and elevation. Occasionally, the river has surrounded such areas, and has completely cut off the natural drainage.

Reclamation Development.—Although the district has been used for agriculture since before the Civil War, it has been handicapped by physical limitations. A pumping project was started within recent years; but while this aided the particular area drained, it raised the water level in adjacent areas because of seepage. Thus, the financial damages prevented the development of this project.

Following this experience, the Cameron County Water Control and Improvement District No. 5 was organized for the purpose of providing irrigation facilities for 23 500 acres. According to Mr. Myers,

"A reconstruction program was begun, a new pumping plant equipped with oil engines was constructed, new canals were built along the banks of the 'resacas' formerly used as main canals, and the canal system was also greatly extended. These canals are still in general use."

Losses from seepage through the banks of the "resacas" continued to be serious. Canals built in the compact soils of the lower area were fairly tight when they became soaked after continuous use; but after being dried, during periods of low water, the banks cracked and the seepage problems arose again. The heavy soils provided poor natural drainage in general, and very soon became badly seeped and salted. Conditions in this district continued to grow steadily worse in the face of these difficulties, until 1926, when the Directors of the District authorized the compilation of a report containing recommendations for correcting them.

New Conditions.—The plan as submitted provided for the construction of 165 miles of open ditch, 57 miles of tile drainage, and considerable length of canal lining, at an estimated cost of $1 100 000. Part of this (about one-half) was covered by a bond issue; hence the work that could be done had to be similarly reduced within the available funds. It was not attempted to drain each tract completely, but rather to make it possible for the property owner to do so if he desired.

In general, open ditches 6 ft. wide on the bottom were standard, with grades varying between a minimum of 0.1 and a maximum of 5.0 ft. per 1 000 ft. Then, again, according to Mr. Myers, tile drains were laid, but most of these, when laid in fine sand, proved of only temporary value.

Draining "Resacas".—One vexing problem, according to Mr. Myers, was involved in eliminating the old stagnant lakes:

"In most instances ditches were dug to completely drain the old 'resacas'. These ditches were cut through the high banks of the 'resacas' and extended up their channels; usually they were shallow and failed to cut completely through the tight alluvial 'resaca' bottoms. In many instances deep pits were dug in the bottom of these ditches in an attempt to reach the underlying sand strata. In some cases where such a stratum was penetrated there was a considerable flow of water, indicating that the ground-water was under pressure.

"In the past the farmers have blamed the 'resacas' with the seeping and salting of their lands. This was probably true where the water level was high. However, the material in the 'resaca' bottoms and part way up the sides, was so tight that it seemed hardly probable that there could have been much seepage from this source at an ordinary water level. The drainage of the 'resacas' will

serve a very useful purpose, as in many cases they are very unsightly and a menace to the health of the community. Often the water is quite shallow and the channels are choked with vegetation. After drainage, the land in the 'resacas' will eventually be capable of cultivation or use as pasture."

In an attempt to measure the effect of the drainage, test holes were sunk throughout the area, about 3 000 in all. Measurements before the ditches were dug, and at intervals afterward, gave an idea of the value of the work. In general, the ground-water was lowered to 5 ft., or more, below the surface; in a few areas it even rose slightly. Eventually, it is hoped that the gradual lowering will provide a flat water-table back from the drains.

Seepage Problems.—Mr. Myers reported varying success in the efforts to prevent seepage. Some of the main canals in sandy soil became tight by silting. Again, sand crabs frequently rendered this lining ineffective. In other instances drainage ditches have been dug on either side of the main canal to prevent damage to adjacent property. Incidentally, this construction tends to increase the seepage losses themselves.

Even a lining of concrete, or the use of concrete pipe, has been tried. One type was concrete lining—a plaster coat consisting of 1½ in. in thickness, placed by hand. It was not reinforced, and had no contraction joints. As a result, it was inclined to crack badly, so that the seepage was considerable. Attempts to repair these cracks with hot asphalt proved difficult.

More recently canals have been lined with 1-in. layers of concrete, reinforced with a No. 10 rectangular mesh, with wires spaced 4 in. apart longitudinally and 8 in. transversely. The lining is done by means of a cement gun using a very lean (1:4) mix.

It is recorded by Mr. Myers that: "This lean mix reduces cracking to a minimum. What cracks have occurred have been transverse and have not opened up sufficiently to allow appreciable seepage."

Success of District.—The result in the El Jardin District has been a remarkable improvement in the general appearance as well as a decided betterment in the morale of the farmers and land owners. Many farms that would have been abandoned, are now being cultivated again, and it is reasonable to expect that the entire district will be under cultivation in a few years. Indeed, "The use of water is increasing and the land owners are paying up their back taxes and flat rates."

DISCUSSION

What is the chemical quality of the water in the Rio Grande? To this question, by Chairman Lippincott, Ned H. Sayford, M. Am. Soc. C. E., replied that there were 7 or 8 parts of totally soluble salts.

In reply to an inquiry, Mr. Myers declared that, in regard to the smooth finish placed on the gunite construction, this work was satisfactory because great care was taken to apply a very light trowel finish "right behind the gun". While it is unusual to finish gunite work, this job was carefully done so as not to disturb the bond.

Comments on the Gunite Process.—Smoothing the surface with a steel brush was recommended by Fred C. Scobey, M. Am. Soc. C. E., who declared that by this means the coefficient for Kutter's formula could be reduced to 0.015. The best lining in the western part of the United States is that of the Ridenbaugh Canal. Five or six sets of tests on this canal have determined a value of 0.013 ±, for the Kutter coefficient.

In reply to a question from the floor, Mr. Scobey stated that,

"The two highest grades we have now of concrete pipe appear to be about the same in carrying capacity; one type is that of the centrifugal cast pipe. That process results in an excellent surface. The method of joining that pipe does not give the perfection of transmission from one unit to the next that we secure in unit lengths that are poured; that is, in the best process of steel on steel."

Referring to the chemical property of water in the Rio Grande, Fred B. Porter, Assoc. M. Am. Soc. C. E., mentioned the effects of some of the contributory streams. The Pecos River brings very bad water, and the Del Rio, on the other side of the border, contributes very good water. Chairman Lippincott added that it seems that in certain places the water in the Rio Grande is as good as that which comes from the High Sierras, and Mr. Myers added that, when the water is low, it is very bad, but, in its high stages, it has very little salt.

A paper by R. G. Hemphill, Assoc. M. Am. Soc. C. E., entitled "Silting and Life of Southwestern Reservoirs", was presented by Mr. Hemphill, but is not available for abstracting at this time.

PROBLEMS OF THE INTERNATIONAL WATER BOUNDARY COMMISSION

By L. M. Lawson,* M. Am. Soc. C. E.

Organization and Duties.—Five boundary treaties between the United States and Mexico serve as a guide to the activities of the International Boundary Commission. The total length of the boundary is 2 013 miles, 1 300 miles of which are marked by the Rio Grande, 20 miles by the Colorado River, and 693 miles by monuments and other land marks between El Paso, Tex., and San Diego, Calif. The Commission is empowered to carry out the provisions of the Convention and Treaties of 1848, 1853, 1882, 1884, 1889, and 1905. It may suspend the construction of any work along the boundary that is contrary to the intent of existing treaties; it may erect and maintain necessary monuments marking the boundary, etc.

The Commission is composed of one representative from each country, and if both Commissioners agree to a decision on matters coming under their jurisdiction, their judgment is binding on both Governments, unless one Government disapproves it within thirty days.

* Commr. of the United States, International Boundary Comm., El Paso, Tex.

Boundary Changes.—In treating at length the history of boundary treaties with Mexico, Mr. Lawson paid tribute to Maj. William H. Emory, as one of the foremost engineers active in solving boundary disputes, remarking,

"His report dated 1857, printed as a document of the House of Representatives of the Thirty-fourth Congress, is a complete compilation of geographical, geological, and settlement conditions existing along the entire border. In July, 1882, a treaty was concluded, providing for replacement of destroyed or removed monuments and for general reconnaissance of the frontier line to be made by each Government."

One problem that has always been especially difficult has been the determination of the ownership of land when a river which forms a boundary changes its course. The Treaty of 1882, among other things,

"Provided that any other change wrought by the force of the current, whether by the cutting of a new bed, or when there is more than one channel by the deeping of another channel than that which marked the boundary at the time of the survey made under the Treaties of 1848 and 1853, shall produce no change in the dividing line, but the line then fixed shall continue to follow the middle of the original channel bed, even though this should become wholly dry or be obstructed by deposits."

When a river boundary leaves its old banks and seeks a new channel, the areas that are thus transferred across the river are known locally as "bancos". In 1905, the treaty provided for the elimination of such "banco" tracts as had been caused by changes in the Rio Grande. Detached tracts with an area of more than 250 hectares (620 acres), or a population of more than 200, remained unchanged in nationalities, according to provisions of previous treaties; but the new treaty provided that:

"The citizens of either of the two countries who by virtue of the stipulations of the convention shall in the future be located on the land of the other, may remain thereon or remove at any time to whatever place may suit them, or keep the property which they possess in the territory or dispose of it. Those who prefer to remain on the eliminated bancos may either preserve the title and rights of citizenship of the country to which the said bancos formerly belonged, or acquire the nationality of the country to which they will belong in the future. Property of all kinds situated on the said bancos shall be inviolably respected, and the present owners and heirs and those who may subsequently acquire the property legally, shall enjoy as complete security with respect thereto as if it belonged to citizens of the country where it is situated."

Bancos.—Under the provisions of the Treaty of 1905, the International Boundary Commission decided the nationality of fifty-eight separate tracts of land located in the storage between Roma, Tex., and the Gulf of Mexico. To-day, 133 bancos representing 17 235 acres, have been transferred from one country to another. Of this total area, 10 384 acres have come to the United States, and 6 851 acres have gone to Mexico. The average area of these bancos was 129 acres. One large tract of 13 000 acres, known as the San Elizario Island, does not come under the provision of the Treaty of 1905. Although this area is now south of the Rio Grande, it is United States territory.

New Agreement and Plan.—Describing the Rio Grande, Mr. Lawson stated that there were three alluvial valleys, the El Paso-Juarez, the Presidio-Ojinaga, and the Lower Rio Grande, in which,

"Numerous river changes bring about boundary disputes. If and when storage gives controlled flows through these valleys, a plan of river straightening will become feasible which will eliminate the causes which give rise to the present difficulties of alignment, enforcement of National laws, and controversies over private titles. This more ideal situation has been brought about with respect to the El Paso-Juarez Valley, and both Governments have long been sensible to the problem arising from the meandering channel of the Rio Grande which generally forms the boundary line, the menace of floods, and other difficulties in an area between El Paso, Tex., and the Box Canyon below Fort Quitman, Tex., a distance by river of about 155 miles. Under special appropriation the countries have instructed their respective Commissioners to proceed with a remedial plan, which would have as its object the stabilization of the boundary line and the rectification of the Rio Grande in the area under consideration."

The land along the Rio Grande is becoming more and more thickly settled and is being used for the cultivation of crops, thanks to extensive National and private irrigation and drainage systems. These improvements create a demand in both countries for

"A more stabilized and protected channel than the meandering and menacing river. The treaties now existing and in force provide for the determination of sovereignty of tracts of land segregated from one country to the other by the natural forces of the river, but do not take into consideration or permit artificial rectification and straightening of the meandering channel necessary in the portion of the Rio Grande being considered."

The City and County of El Paso face these problems especially. In the past few years they have expended $500 000 in the protection of city and county lands. More than $7 000 000 have been expended in constructing irrigation and drainage works, with an additional equal amount in preparing lands for cultivation. Mr. Lawson treated at length the influences to be expected by the construction of these drainage works, such as the Elephant Butte Project. These projects include elaborate plans for stabilizing the flow in controlling the direction of flow of the boundary rivers.

International Co-Operation.—Quoting directly from Mr. Lawson's paper,

"Both sections of the International Boundary Commission, aided by important and valuable suggestions and constructive comment from consulting engineers, have agreed upon and recommended to the two Governments the preliminary and general features of such a plan, and are now developing final designs and estimates.

"In the routine business of the Commission, now organized upon an engineering basis, covering action on various land changes and particularly in the field and office engineering work in developing the plan as outlined, the co-operative and helpful spirit of the Mexican Section is a matter for special note. The personnel of that Section of the Commission has made possible unified and constructive action on boundary matters."

DISCUSSION

In discussing this paper, N. T. Veatch, Jr., M. Am. Soc. C. E., called attention to the fact that a project which required the straightening of a stream 180 miles long, so that its length is 80 miles, is one of major proportions. The

Army Engineers have a theory that when a stream is shortened in this way, it immediately begins to try to recover its original position.

Replying to an inquiry as to whether this feature had been considered in connection with the present case, Mr. Lawson stated that in the section of the river thus shortened, there was available a storage of water from Elephant Butte Dam. The changes made to the river sections at the points in question simply involved putting them on self-cleaning grades.

Silting of Lake Austin.—In discussing Mr. Hemphill's paper, supplementary data concerning conditions at Lake Austin were presented in a discussion from T. U. Taylor, M. Am. Soc. C. E., read by John A. Focht, Assoc. M. Am. Soc. C. E. The silt is gradually narrowing the channel, and has reduced it from a width of 1 300 ft. in 1913, to 620 ft. in 1929. Points in the mouth of Bee Creek that were originally (1913) 51 ft. deep, are now filled with silt to a level of 2.4 ft. above the lake level, so that the total depth of silt is 53.4 ft.

SURVEYING AND MAPPING DIVISION

APRIL 25, 1929, 2:30 P. M., TO 5:45 P. M.

"The technical procedure of making surveys in the United States is far behind that of other countries, even of the newer ones, such as South Africa and New Zealand." With these words, Chairman C. V. Hodgson* opened the meeting of the Surveying and Mapping Division. Continuing, he said:

"With the increase in property values, the past few years have emphasized the need for more accurate surveys. The economy is seen on big construction jobs, in spending a few thousand dollars for good surveys, rather than to proceed with construction without adequate information. It is a very expensive proceeding on a construction job to cut steel while it is being put into place, or to correct the alignment of a tunnel after the tunnel headings have met."

Special Committees of the Division are at work, he stated, preparing reports that will outline suggested limits for these conditions. One of these is the Special Committee on Definitions of Engineering Terms. After these preliminaries he introduced the first speaker.

PROBLEMS IN RE-TRACING OLD SURVEY LINES IN TEXAS

By B. F. Williams,† M. Am. Soc. C. E.

Original Marks.—The purpose of this paper was to present the history of the engineering and legal phases of surveying in Texas as they affected the work of the modern surveyor. In the words of Mr. Williams,

"It is a well-fixed principle of law that in relocating surveys it is the duty of the surveyor to follow the boundary lines laid on the ground in the original survey. 'Move not the ancient land marks' is an old legal maxim given by Moses in Deuteronomy. Land boundary litigation has burdened the Civil Court dockets since the beginning of private ownership of the soil, and many wars between kings, princes, barons, and lords have been fought over disputed boundaries.

"Indefiniteness in description and inaccuracy in laying down the original boundaries cause most of such litigation. These deficiencies also are causes of the greatest difficulties to the surveyor in making a relocation. The boundaries of a survey are usually a series of connected straight lines having given lengths and bearings, and designated natural or artificial objects, marks, and monuments; or they follow the meanderings of streams or divides of water-sheds. The natural marks are streams, mounds, hills, rocks, trees,

* Major Hodgson died on May 19, 1929.

† State Reclamation Engr., Austin, Tex.

etc. The artificial are crosses, blazes, hacks, drill holes, stakes, letters, figures, mounds, pits, a pile of buffalo bones, a cow's head, or other conventional signs and characters on standing timber, rocks, ledges, or on the ground. In order to retrace the lines of a survey it is necessary to have the description of the original boundaries made by the surveyor who laid them down. This description, called the field notes, is usually found in the archives of the General Land Office, or in the records of deeds in the county where the survey is located. It is also necessary to know the laws, rules, regulations, and conditions under which the survey was made."

Texas Laws.—The land laws of Texas have felt the influence of Spanish and French laws, which, in turn, have a common origin in the Roman Civil Laws. "After the Texas Revolution in 1836, the principles of English Common Law took precedence over the Spanish and French influences."

In 1845, Texas was admitted into the American Union of States. Quoting Mr. Williams,

"By a provision of the Act of Admission, Texas retained title to all her public vacant land and the State has administered these lands chiefly for the benefit of railroad development and the endowment of a public educational system. The United States has never owned or administered the public lands of the State and, therefore, the United States system of land surveys has never been adopted."

Measuring Units.—One of the most interesting parts of this paper was Mr. Williams' description of land-measuring units that prevailed in early times in Texas. He stated that:

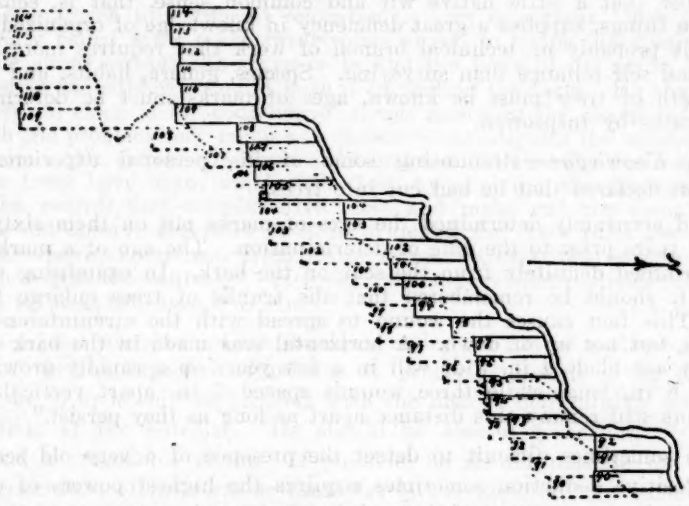
"The Spanish and Mexican law prescribed the units of land measurement and the methods of laying out surveys and marking the lines and corners. The linear unit is the 'vara' (in the Austin collection of documents frequently called 'bar'), equal to 33⅓ in., and defined as 3 geometrical ft., or 'pie'. The vara had different values at different dates and in different countries. When the Americans began to make surveys in Texas each surveyor had his own idea of the length of the vara. Austin prescribed the length at 33⅓ English in., and this value has become the standard. The geometric foot was generally used in the Sixteenth Century. Morgan gives its value as 9.8 in. and Locke as 10 in.

"The units of area are the square pie, square vara, labor, and league or 'sitio'. It takes 5 645.376 sq. varas to make an English acre. A labor contains 1 000 000 sq. varas, or 177.13 acres. It is 1 000 varas square. The league is 5 000 varas square and contains 25 labors, or 4 428 acres. Many of the Spanish surveys contain a league and labor, or 26 labors of 4 605 acres. Five leagues or sitios is a hacienda. The labor in the theory of the Spanish laws and customs is a farm, or an area upon which to labor and to produce crops, in contradistinction to a league upon which to conduct ranching and stock-raising operations."

Value of Markers.—The relative merits of land markers were described, Mr. Williams stating that natural objects received preference over artificial marks or monuments, as well as witnesses to boundaries. At various times, the State Courts have upheld this principle inherited from Spanish laws. Quoting, again,

"In relocating surveys the Courts consider four elements in the following descending order of importance: Natural objects, such as rivers, creeks, mountains, etc.; artificial objects, such as marks on trees, marked lines, stakes, etc; course and distance; and quantity. These rules are founded on

reason, experience, and observation, and the surveyor must approach the problem of relocation with them well in mind, for he may be, and frequently is, required to defend the accuracy of his work before the Courts. While the surveyor must know the law, he has done his whole duty when he has found all the facts relating to the original boundary, relocated it upon the ground, and reported his findings of fact to his principal or to the Court. In many cases he will be unable to harmonize the description given of the original boundaries with the facts he finds on the ground."



SECTION OF CANADIAN RIVER IN TEXAS PANHANDLE: SOLID LINES SHOW LOCATION AS CALLED FOR IN FIELD NOTES; DOTTED LINES SHOW ACTUAL RIVER AND SURVEY AS THEY WOULD LIE IN RELATION TO IT.

Qualifications of Surveyor.—In order to be fully competent, a surveyor must be quite a versatile person. In making investigations of natural objects called for in a description of a boundary, he is confronted,

"With problems in physiography, geology, and ecology. Stream channels change location by erosion, avulsion, and accretion. The Courts have laid down rules of law governing titles to lands affected by such changes, but the surveyor is not concerned with titles. It is his duty to lay down the lines on the identical location of the original boundaries. He must locate and identify ancient stream channels and terraces and determine their ages by strata formations, timber growth, and collateral facts. In my experience I have found river channels ¾ mile from their record locations fifty years before.

"The beds of all navigable streams in the State belong to the public. A statute of 1837 defines a navigable stream as one with a mean width of 30 ft. or more from its mouth throughout. The statutes prescribing methods of laying out surveys prohibit crossing navigable streams. The effect is to leave such stream beds in the public domain. Many streams in Texas are dry throughout a great part of the year, but in wet seasons the wetted perimeters are several hundred feet long and the width between the cut banks of the freshet flows make them navigable under the statutory definition given above. It appears that surveyors understood the definition of a navigable stream to mean a width of water of 30 ft. and gave no importance to the width between the cut banks of the flush flow."

In attempting to locate original boundaries, the surveyor must be qualified to interpret the changes made by the forces of Nature. Furthermore, according to Mr. Williams,

"An investigation of artificial objects requires no less skill and experience. A fair knowledge of botany, plant anatomy and physiology, and ecology is required. Many efficient, capable, and skillful surveyors have no organized knowledge of these subjects and probably have never heard or seen the names of these sciences; they are students in them nevertheless. We should remember that a little native wit and common sense, that is, sense about common things, supplies a great deficiency in knowledge of organized science. There is probably no technical branch of work that requires more common sense and self reliance than surveying. Species, genera, habits, and methods of growth of trees must be known, ages of marks must be determined in many cases by inspection."

Tree Knowledge.—Recounting some of his personal experiences, Mr. Williams declared that he had cut into trees,

"And accurately determined the ages of marks put on them sixty-five to seventy years prior to the date of determination. The age of a mark cannot be determined definitely from the scar on the bark. In examining the bark wound it should be remembered that the trunks of trees enlarge laterally only. This fact causes the wound to spread with the circumference as it enlarges, but not up or down. A horizontal scar made in the bark with an ordinary axe blade 4 in. wide will in a few years on a rapidly growing tree become 8 in. long, while three wounds spaced 6 in. apart vertically on a tree trunk will remain this distance apart as long as they persist."

It is sometimes difficult to detect the presence of a very old scar on a tree. Positive deduction sometimes requires the highest powers of observation; but there are means of determining from such scars several important facts concerning the original survey. For example, to quote Mr. Williams,

"The field notes describing a boundary usually give the sizes and genera of the trees called for, and, many times, timber lines and character of flora. While magnitude of size varies directly with magnitude of age, the surveyor must not assume that all large trees are very old, or that all small ones are very young, and refuse to examine small trees for old marks. In 1920, in making a relocation, I found a cedar elm (*Ulmus classifolia*) which was marked in 1842 as a witness to a corner. The locating surveyor gave the size as 6 in. in diameter. In 1920, the diameter was 8 in. and after the moss was scraped off, the marks were perfectly plain."

Recovering Timber Lines.—Old surveyors often describe timber lines as natural boundaries. In attempting to relocate old surveys, the modern surveyor often finds that the timber has been cut and if it has not been entirely cleared, the present line differs considerably from the original. Excavations can be made, however, to determine the old line, and, according to Mr. Williams,

"To make the investigation pits are dug 2 or 3 ft. in the ground. By careful examination of the vertical banks of these pits the forms of tree roots can be traced by the presence of root bark or coloration of the earth long after all trace of the wood fiber of the roots has perished. Most hardwood trees have large tap roots, penetrating deep into the substratum of clay. These roots persist long after all surface traces of the trees are gone, and even after these roots have completely decayed their forms can be easily

traced by the residual humus and the surface soil washed and settled into their clay moulds.

"My experience indicates that the pioneer surveyors ran most of their lines long, and that the bearings vary on the ground considerably from those given in the field notes; but in making a relocation of an old survey, if the surveyor will use a light, open-sight compass with an ordinary Jacob staff he will have little difficulty in following the marked line."

Duties to Clients.—Quoting further, in regard to this subject,

"A proper approach to a difficult relocation is very important. The surveyor is usually called after a controversy has arisen to which his client is a party. The client expects the surveyor to find the facts in his favor, but the only interest the surveyor should have is in finding the facts. He should make his position clear to the contending parties and with the record before him approach the problem very much as a detective investigates the mysteries of a great crime. He should run down clues and test out theories until some one or more facts have been established, then, by applying the descriptive matter of the record, find corroborative facts and make out the complete case. He should observe the rules of legal evidence, for these rules are the results of our race's experiences from times so remote that 'the memory of man runneth not to the contrary'. Hearsay testimony without corroborative facts cannot be accepted, except as clues."

Although a land surveyor may be working under the pay of a client who hopes to profit by the expected adjustment in property lines, it is the duty of the surveyor to approach the problem with the idea of determining actual facts regardless of the outcome. He should be absolutely certain of his results beyond a reasonable doubt. If possible, he should relocate the old boundaries regardless of interests or conflicts; but if facts are not sufficient to relocate these boundaries, he should report to his client the conflicting evidences, giving that client the benefit of all clues, theories, and facts found during the investigation. Doubtful relocations can be settled by arguments in Court. In conclusion, Mr. Williams re-stated the duty of the surveyor, as follows:

"Therefore, it is the duty of the surveyor to find and identify the objects called for and lay the relocated boundary on the ground accordingly. The surveyor is employed to find and report the facts. His duties do not require him to become an advocate of his principal, but it is his privilege and duty to defend the facts set out in his report."

DISCUSSION

OTHER SURVEYING PROBLEMS IN TEXAS

By J. M. Howe,* M. Am. Soc. C. E.

Peculiarities of Texas Surveys.—Recommending that engineers from other districts planning work in Texas should trust the surveying part to local surveyors, Mr. Howe gave as his reason that,

* Cons. Engr. (Howe & Wise), Houston, Tex.

"Texas land line location is so interwoven with Texas land law that the surveyor or engineer who depends upon the rules of the United States General Land Office will soon be completely lost. Some one has said that 'land surveying is an art and not a science', and if this applies anywhere, it certainly does in Texas."

Modern difficulties attendant on the surveying of lands are the result of natural increases in land values. In fact, according to Mr. Howe,

"The land when the first surveys were made, had values of less than $1 per acre, and not much more than for the later fill-in tracts. In consequence, chaining was carelessly done and the course was only approximate.

"Texas law requires that precedence be given to senior surveys, and many of the conflicts in title are due to failure or inability of the later surveyors to identify the earlier lines. Theoretically, the surveyor should be able to re-locate the surveys by giving the senior surveys their distances and thus replace on the ground the entire map structure; but original monuments, when identified, hold over both course and distance, so that, regardless of the recorded length of lines, when junior surveys are in conflict, the junior must give way. When one appreciates that the early surveyors were working for the grantee and not for the sovereignty of the soil, he can readily understand that many of the early grants have lines on the ground considerably longer than the distances called for in the patent, and it is these lines that hold and not the ones called for in the field notes."

Conflicts of Errors.—Unfortunately, errors in bearings are as prevalent as those in distance. In Texas, observed Mr. Howe, north is seldom true north. Most of the early surveyors knew that the magnetic needle had a declination, but their ideas varied considerably as to its value. Surveys of identical lines in the same county have been known to vary as much as 5° in bearing. Quoting, further,

"When the work of different surveyors comes together, the result is what a facetious Judge once termed the 'Texas rectangle', which he defined as a 'rectangular parallelogram, no two sides of which are the same length and no angle of which is a right angle'. Apparently, there was never any attempt to allow for earth curvature, and the change in length between meridians due to curvature has to be absorbed in errors in course or distance. This, however, is a minor matter, as the errors in execution of the work are sufficient to make curvature corrections negligible."

Surveyor or Lawyer?—Mr. Howe agreed with Mr. Williams that the duty of the surveyor is to find and report the facts, and that he is not required to become an advocate, provided the surveyor is working for a State or is qualifying as a witness. On the other hand, when he is employed by a land owner, he will be expected to outline the property and report to the owner as to how many acres he has a title, and just what lines he can actually claim. The surveyor, therefore, must have a working knowledge of land laws. He must be qualified to decide in the field whether he should follow a well marked and apparently well recognized crooked line, or connect two corners with a straight line. When "calls" are inconsistent, he must know which one he is entitled to change. He must have a knowledge of limitation laws in determining what weight to give to existing fence lines as property boundaries. Concluding, Mr. Howe stated:

"The importance of fencing and consequent limitation, is a constant source of trouble to the surveyor, but limitation, in principle, more than com-

pensates for the immediate inconvenience in that it is definitely fixing the lines of surveys. With established limitation lines the theoretical lines lose their importance and surveys become definitely solvable mathematical problems.

"The lot of the Texas surveyor is not an easy one. He must be both lawyer and surveyor, as well as a fair prophet as to how the Courts will construe his decisions. The responsibilities are out of all proportion to the compensation, with the result that the younger men are reluctant to take up the vocation. The fixing of lines by limitation and the loss of the earlier witness trees through decay, are rapidly doing away with the need for the old-time land surveyor, and his place will be taken by the engineer. This is probably just as well, for, like the railroad locating engineer, the need for the artistry of land surveying is largely in the past."

GENERAL DISCUSSION

W. N. Brown, M. Am. Soc. C. E., asked if there was any State legislation that required proper demarcation when old grant lines are determined. He was told there was none, and that the difficulty arose because the fixing of the land line is usually a matter between litigants and, therefore, not binding on owners on the other side of the survey. Thus, the State cannot touch the interest of any owner who is not a party to the litigation.

One solution, according to Mr. Williams, was to build a fence anywhere on the land, and if it remains there for certain periods of time according to the particular statute affected, it becomes controlling. In one case, the owners built their fence line so that 20 ft. of their land was on the outside. The adjoining owners connected their fences to the first one and the resulting boundary has now become fixed beyond the power of change.

In the opinion of D. L. Lewis, Assoc. M. Am. Soc. C. E., surveys in Texas are now less inaccurate than Federal surveys. The surveys made by the old pioneers with the open-sight compass were remarkably accurate considering the difficulties involved. He declared he had been successful in following old land surveys establishing lines between Kansas, Arkansas, and New Mexico, and it was his belief that in some respects the old surveyor was as good as some of the present theoretical men. In the eastern and northeastern parts of the country, many private surveyors claim that they are correcting all the county surveys to the State and National triangulation points, and making these corrections in the terms of co-ordinates and part of the legal record.

Chairman Hodgson pointed out that,

"The surveyor cannot change the State lines regarding title rights. He can, perhaps, by his methods of recording new surveys, make it easier for the next man to find those corners. * * * In South Africa the law is that no title can be given unless the records of that survey contain the co-ordinates of the National Triangulation System of third-order accuracy."

The Chairman then called attention to a statement made by Victor Gelineau, M. Am. Soc. C. E., that the value of land in New Jersey had now reached so high a price that third-order accuracy was not sufficient. The surveyor was advised to look forward to an increasing demand for second-order accuracy.

Reverting to the question of the old land surveyor, Mr. Williams declared that in locating land on the Canadian River, a surveyor wrote the field notes, submitted them to the General Land Office, and they were approved and the land was granted. During a subsequent lawsuit, the surveyor testified that the only field work done was the taking of two bearings on the course of the river, while an assistant kept watch at the top of the cliff to give warning of Indians. After taking the two sections on the course of the river, and assuming that it continued indefinitely in the same course, he went into the safety of his office and wrote out the field notes for 116 sections of land.

In the words of Mr. Williams, "when asked why he did this, he said 'It would have taken a company of U. S. soldiers to have kept the Indians off me while making these surveys, and the land was not worth it'."

AIRPLANE SURVEYS FOR PIPE LINE AND TRANSMISSION LINE LOCATION

By Stuart Moir,* Esq.

The major usefulness of airplane surveying, according to Mr. Moir, is in large projects such as the Antarctic Exploration conducted by Commander Byrd, and the Patrick Exploration in Rhodesia which involves the mapping of 50 000 or 60 000 sq. miles of land for colonization purposes. In Canada, there are under way extensive exploration projects for the study of hydro-electric development and for the location of precious metals. Such surveys are also proving successful for the location of transmission lines in various parts of the United States and Canada.

Progress of Aviation.—The industry of aerial mapping has become commercially important within the past ten years. In location and reconnaissance surveys for transmission lines and pipe lines, there are considerable advantages to be gained. In the words of Mr. Moir:

"It has been said that the three major factors in the selection of a route for a transmission line are:

1.—Electrical features;
2.—Mechanical and civil features; and
3.—Real estate features.

Of these, perhaps the real estate feature should be given the greatest consideration, because if it is neglected, it will involve a greater waste of money than either of the others."

This feature involves many problems that influence the cost of construction. Property values must be considered, and because of peculiarities of some property owners such values may vary most unexpectedly, so that very often a detour may require a sudden change of plans.

Value of Reconnaissance.—As Mr. Moir remarked:

"Experience shows that the cheapest lines are those which have had the greatest amount of reconnaissance work. Hasty decisions to construct lines

* Dallas, Tex.

have frequently caused engineers to rush field work, and right-of-way departments have been obliged to purchase property without due consideration to property trends. The obstacles to be overcome are frequently costly from the 'dollars and cents' viewpoint, not to mention strained public relations."

The aerial map offers a means of relieving the situation. By its proper use a great many obstacles can be anticipated and preliminary measures can be taken to surmount them.

Methods of Procedure.—Mr. Moir described in detail methods used for photographing the proposed route of the transmission line. For example,

"All roads along the proposed route of the transmission line are traversed by competent field men, taking notes on unusual field conditions. They obtain all the information possible about local people, and all the available data regarding the country are assembled. Property holders are listed and special attention is paid to individuals who might oppose the line. In the meantime, Court-house deed searchers can be obtaining copies of the deeds of property. This deed data are all assembled and plotted on to the photographic map, thus showing the bounds and ownership of every parcel of property which the transmission line is to cross.

"An intensive study of the aerial map now can be made to select the best route for the line. It is at this point that an aerial map is perhaps of greatest value in transmission line work, for it permits one to have a picture of the line in its entirety, to aid in making decisions and selections. The engineer can readily balance the advantage or disadvantage of a detour in respect to the whole line. In other words, he has before him, at all times, a picture of the entire project without which it is extremely difficult to visualize the true evaluation of detour in the line."

Use of Aerial Map in the Field.—The next step is to provide copies of the aerial map in sections so that they can be folded and filled out. These are then taken into the field so that the tentative route can be thoroughly and completely inspected to study construction features. For instance, according to Mr. Moir,

"Experience has shown that there are many cases where swamp foundation construction is not nearly as expensive as the construction department will insist, and it is often cheaper than purchasing adjacent upland property. All these factors must be balanced, one against the other, and with an aerial map in the field, the reconnaissance engineer can sketch on to it, the limits within which the route can be re-arranged to take full advantage of construction and real estate features."

Completing Map.—The next step is to bring the map into the office where the necessary corrections are made on the office map. The aerial map is especially valuable in making deed adjustments, because such a map will often show a brook, a row of trees, a stone fence, or other natural objects, which are described in the deed. Then,

"After all the property data have been plotted, the right-of-way center line can be ruled in, also the lines showing the width of the right of way. Tracings are then turned over to the real estate department to be used in negotiating with the property owners."

With this map completed, real estate men will be in a position to discuss individual problems with land owners, and when the right of way has been fairly well determined, work can proceed in running profiles, locating towers, or poles, etc.

Time-Saving Attributes of Aerial Mapping.—The use of an aerial map can effect a saving in time of 25 to 50% of that required by means of complete ground methods. Mr. Moir cited,

"One very striking instance of a 40-mile line, on which at least one month's time of a surveying party was saved, and the work of title examination reduced about 25 per cent. In a congested territory, the property values are very high—$60 000 per mile was the cost of right of way alone. A 2-day study of the aerial map was more effective in laying this line, than a month's study on the ground would have been. At least one-half the ground surveying for the line was eliminated through the use of the aerial survey.

"In the matter of title examination comparing work where no aerial map was used with one on which it was used—results indicate that the saving amounts to about 50 per cent. There is a psychological advantage of quick right-of-way work which cannot be accurately estimated."

Accuracy.—Declaring that such maps are accurate and reliable, Mr. Moir pointed out that:

"Accuracy is relative only. As you all know, the cost of increasing accuracy, both in field work and in plotting, probably rises as the square of the increase, so that, in all survey work, it is necessary to decide what limit of accuracy is required so as to keep down the costs as much as possible. In the congested areas of our country, aerial maps with a maximum error of 1% may be required. On the other hand, in the less populated regions, such refinement is not necessary, and a map made from uncorrected photographs serves the purpose quite as well as the more refined products. In aerial surveying, as in ground-surveying operations, instrumental errors of every kind must be guarded against. The aerial method, however, being largely a mechanical one, is free from the human blunders so common in ordinary surveying."

Some measure of control is obtained through the relative scale desired. This will vary with the type of the country to be mapped, the details required, and the money available. In densely populated areas, Mr. Moir recommends a scale of 1 in. equal to 400 ft. In open country, a scale of 800 ft. to the inch should be sufficient, and in the arid Western United States, one of 1 200 ft. will be found ample for field purposes.

The Stereoscope.—A splendid idea of relative differences of elevation may be obtained by exaggerating the vertical scale by placing two overlapping photographs in proper position under the reflected mirrors of a stereoscope. This enables the engineer to obtain a complete view of the topography of the country. It is also possible to check the accuracy of the contour lines sketched in the field by this method. In brief, the stereoscope has proved so valuable that it is now considered an essential part of the equipment in both transmission and pipe-line surveys.

Pipe-Line Location.—The advantage of airplane surveying for the location of pipe lines is in the time saved, the publicity avoided, and the right of way acquired with the least amount of public resistance. Errors of judgment on the part of a locating engineer can be avoided, Mr. Moir stated, by using "a method which pictures every foot of ground to be covered by the proposed line." In fact,

"In addition to being of value to the locating engineer, the photographic map reveals to the construction engineer, and to the maintenance superintendent, information of which they would remain in ignorance without its

assistance. Material savings are effected in initial construction and hazards of maintenance and construction reduced to a minimum. A saving in the length of the line, as well as the price of right of way may reasonably be expected. Experience has shown that where field parties have failed to locate the best opening in rough country through which to take a line, the aerial map has revealed the one which is most advantageous for its location. It is frequently advisable for the locating engineer to make a previous observation flight, with the photographic map crew, over the area to be surveyed. Owing to the inaccuracies of many existing line maps, this prevents the possibilities of the pilot getting off his course. Furthermore, the engineer obtains a better grasp of his problem of location and the particular requirements of his line."

Incidental Helps.—Summarizing the advantages gained, Mr. Moir declared:

"The photographic map furnishes a wealth of detail, such as physical obstacles which must be avoided, industrial developments, difficult stream crossings, or hazardous swamp areas. The permanence of the selected location may be judged from a close study of the surface drainage. Firm river banks, loose surface soil, and forested areas, each records photographically with individually distinctive character. The pipe-line right-of-way man derives the same benefits from the aerial map as the transmission-line engineer in obtaining properties for the necessary right of way."

In conclusion Mr. Moir called attention to the fact that in Canada large trees are often used as corner posts, and that woodsmen have become adept in finding these trees with the help of airplane photographs. With the aerial survey, "it is possible to pick up old roads that have been forgotten, and with a little brushing up of the old lines, put them to use." The aerial surveying industry has developed so that the best and most adaptable features of photography and the manufacture of cameras as well as the manufacture and design of special aircrafts can be easily controlled.

DISCUSSION

AIRPLANE SURVEYS

By W. N. Brown,* M. Am. Soc. C. E.

The value of using aerial maps in connection with topographic surveys was emphasized by Mr. Brown:

"Its principal advantages are speed of accomplishment, and wealth of detail that is shown on the produced maps. The disadvantage is the difficulty of maintaining accurately a fixed scale."

Accuracy.—Advocates should not unthinkingly make statements that this method is more or less costly than other methods, because the economies will vary with the particular work to be undertaken. Referring to accuracy of scale, there is still much room for improvement. As for maintaining a permissible error of 1% in congested areas, it was Mr. Brown's opinion that this value,

* (W. N. Brown, Inc.), Washington, D. C.

"Is given as the best that can be expected under favorable conditions. Where the area is hilly or mountainous, the distortions within the individual photographs will be considerably greater so that an accumulation of error varying from 50 to 100 ft. per mile may be expected. Unless this condition can be remedied aerial surveys are only good for reconnaissance purposes. The surveyor of 100 years ago had an advantage here for he had no trouble in keeping his errors down to 5 or 10 ft. per mile. The modern surveyor has a low standard indeed that allows errors greater than 1 ft. per mile."

There are two distinct spheres of activity in this line of work: One, that of the engineer, and the other, that of the photographer. Many failures in the past have been due to not recognizing the division of the work between these two. The adaptation of this method of surveying to topographical work offers many advantages, especially on large areas.

Procedure.—An outline of the method of procedure, as given by Mr. Brown, was as follows:

"First, a controlled mosaic is made on plane-table sheets, 24 by 30 in. in size. In order to get an accurate control mosaic, there must be located over the area a number of points that show on the photograph. Co-ordinate positions of these points must be computed and the points platted on the plane-table sheets. The photographs are then brought to proper scale in agreement with these platted points, and made into a photographic base map, upon which the co-ordinate lines are drawn. The plane-table sheet is then ready for the field where the contours are located by stadia vertical angle methods.

"The topographer starting from a known control point, carries a continuous traverse for position and elevation, at the same time checking the accuracy of his photographic base map. This map also serves as a check against mis-read or mis-platted rods. If he reads a rod at a stream fork or fence line, or other definite feature, and it does not agree in position with that shown on the base map, he checks his reading and either discovers it to be his error or picks up an error in his base map. If the photographic base map has been properly controlled and made, the error will usually prove to be his own.

"The economy of field operation is apparent. He does not have to read rod, for such details as stream and road location, buildings, or other objects, but only takes rod readings necessary for controlling the contour location, and checking the base map where he thinks it advisable. The field work is reduced at least 50 per cent. The resulting map has better expression of contours and details of stream than could be obtained for the same money by any other method.

"The error of scaling the distance between any two points located at widely separated distances on the map should not exceed 1 part in 5 000."

GENERAL DISCUSSION

Aerial Surveys for Transmission Line Location.—Mr. J. B. Thomas, of the Texas Power and Light Company, Dallas, Tex., contributed an interesting discussion of practice in Texas as regards the location of transmission routes by aerial surveying. This system has been in force for about five years, during which time from 300 to 1 000 miles of transmission line have been built each year.

In Texas, the relative change in altitude is small, and it was found unnecessary to go to the extreme refinements of establishing ground control and

making detailed photographic adjustment to eliminate errors in altitude, etc. Pictures are taken on a scale of about 1 in. to 700 ft. along the proposed transmission-line route, so that the stretch photographed is approximately a mile wide. In addition to the airplane photographs, it was necessary to make reconnaissance by combining all the available maps so as to get general contours of the country. This map was given to the aerial surveyor who flies over the area at a height of about 3 000 to 5 000 ft., and sketches what appears from visual observation to be the preferable route. Then, the photographer flies with the pilot and takes pictures of the entire proposed route. The photograph is then converted into a simple mosaic mounted on stiff board, so that various parts can be matched end to end, in order to give the entire length of the line.

The entire map is pieced together in this way, with markers at the angles and along the lines tentatively chosen for the route. Then, every one interested in the project and in the survey is called in to criticize and suggest improvements. The result, according to Mr. Thomas, is very satisfactory. Finally, a line tracing is made of the location adopted, and this is sent to the field and the line is accordingly staked out.

Limitations.—Others who have attempted to follow this method have met with varying degrees of success, and a failure was usually caused by lack of careful preparation. Mr. Thomas declared that he had found aerial photography useful in the study of the growth of towns, in the development of fuel areas, and in surveys of hydro-electric properties. However,

"It is not a cure-all and it is not a substitute for all other kinds of mapping, but it is a method which, when carefully used, after forethought and the exercise of good judgment, can be very helpful to the engineer whose responsibility is to locate such details of construction as transmission lines, and, no doubt, oil pipe lines."

In two or three days of careful study of the photographs, a man may become quite familiar with the character of the country, but, according to Mr. Thomas, it takes some time to learn to read photographs accurately and with judgment. He found that a new man can be broken in very quickly.

Records for Future Use.—Sometimes the work for which an airplane survey is originally intended does not require elaborate ground-control surveys. The result is that, in later years, when it becomes desirable to use the airplane surveys for more extensive work, they are rejected because they cannot be tied in with permanent monuments on the ground. It was suggested that perhaps it would be possible to simply mark out permanent points on the ground that would become recorded on the photographs. Later, these points could be located for the ground-control survey if work of such importance as to justify the added expense came under consideration.

In reply to a question by Mr. Moir, Mr. Brown stated that in plane-table work the field cost could be reduced fully 50% by the use of supplementary airplane surveying.

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PAPERS AND DISCUSSIONS

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SILT TRANSPORTATION BY SACRAMENTO AND COLORADO RIVERS AND BY THE IMPERIAL CANAL

By C. E. Grunsky,* Past-President, Am. Soc. C. E.

Synopsis

Data relating to the transportation of silt or fine sand and other detrital material by the Sacramento and Colorado Rivers and by the Imperial Canal, with inclusion of the results of personal observations, are presented in this paper.

Repeated surveys to determine the frequent changes in the elevation of the bed of the Imperial Canal in its upper reaches, and the removal of large quantities of silt from the head reaches, have furnished an exceptional opportunity for estimating the volume of silt transported by the canal, including its bed load. Over a period of 2½ years the available data indicate deposit followed by erosion to have averaged in excess of 500 000 cu. yd. per month, with a canal flow of about 2 000 000 acre-ft. per year.

The suspended load of silt in the Colorado River has been determined with unusual care at Yuma, Ariz., by the U. S. Bureau of Reclamation, and an analysis of the observations does much to explain the distribution of silt throughout the cross-section of the river, the varying quantities of silt at different stages of the river and at different times of the year, and the total volume transported during the period of observation, since 1909.†

Observations on Sacramento River, California

Early Gauging.—In 1879, one of the gauging stations on Sacramento River, was located at Freeport, Calif., about 14 miles below Sacramento. For this station a program was arranged by the State Engineer, for a compre-

Note.—Written discussion on this paper will be closed in December, 1929.

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

† The writer was on the Engineering Staff of the State Engineer of California from 1878 to 1888. He was a Consulting Engineer to the Imperial Irrigation District from 1916 to 1920.

hensive observation of phenomena throughout a fairly long stretch of river, to develop a dependable rating curve and to facilitate computation of discharge for any known river stage.

The instructions covered a careful determination of the river cross-section in order that the necessity for sounding might be avoided when gaugings were repeated. Additional cross-sections 100 ft. above and 100 ft. below the section selected for gauging with current meters, were to aid in establishing the average river dimensions over a 200-ft. float course. The gaugings were made at a fairly high stage (15 to 20 ft. above low water) and velocities were measured both with current meters and with floats.

The equipment included a catamaran, constructed of two rowboats about 2 ft. apart, united by a light platform. On this deck there was a simple structure that supported a drum of small diameter which was turned with a hand-crank and was used for lowering and raising a weighted frame that carried the current meter. With this outfit there was no trouble in lowering the meter to any desired depth and holding it there. A stay-rope from a boat 100 ft. up stream held the meter frame vertically beneath the catamaran. In taking soundings the depth was read accurately on one of the two light cables by which the frame was raised and lowered.*

The work brought its disappointments, however. There was no agreement between the original and later soundings. Evidently, something was happening to the bed of the river, and, therefore, a systematic study was undertaken to determine the extent of the changes. From day to day soundings were taken in longitudinal lines about 50 ft. apart, each carefully located. At the upper end of each line the control boat was secured by two anchors, and its position was definitely determined by triangulation from shore. A second boat was then dropped down stream and carefully held to line by a helmsman, in order to take soundings every 25 ft. The stretch of river thus covered extended 400 ft. up stream and 100 ft. down stream from the gauging section.

Hydraulic Gradients.—The cross-sections in Fig. 1 show that the configuration of the river bottom, which is a straight reach at this point, was uneven and constantly changing. The material in the river bed was clean sand, most of which was fine enough to pass a 40-mesh screen.†

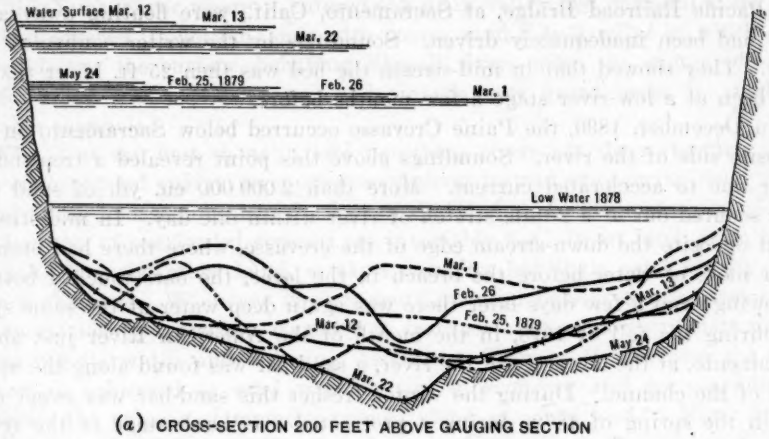
The gauging work was intended not only to measure discharge, but to establish a dependable value of n in Kutter's formula. Consequently, a careful observation of the river slope was to be made. Gauge rods were set at the gauging station and at points about 4 000 ft. up stream and a like distance down stream and, for days, readings of the water-surface elevations at these three points were taken at 10-min. intervals.

When the observations were plotted on a time scale it was seen at once that the water surface gradient was constantly changing. There was a rather long rythmic pulsation as if elongated waves were following each other down the river. It was found, in other words, that simultaneous observations would not correctly represent the river gradient, but that this gradient would have

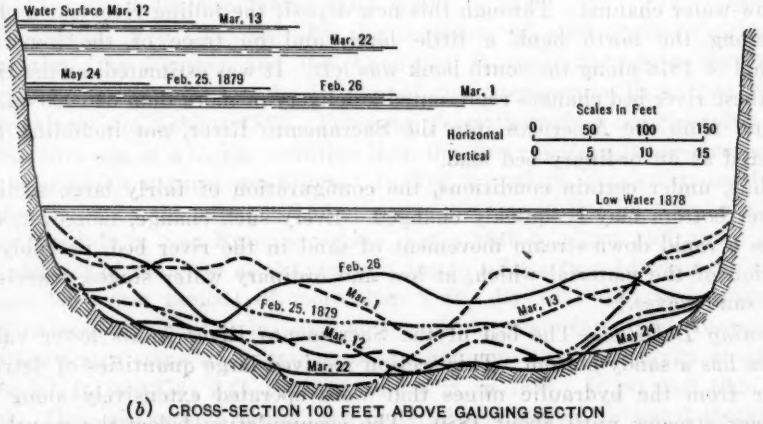
* See Report of Commr. of Public Works, California, 1895. Contribution by the writer.
† This is the writer's estimate; no actual determination was made.

to be determined from a long series of observations. The cause of the pulsation was not discovered. In part at least it may have been the result of sudden changes in the river bed.

River Eccentricities.—An anchored rowboat, unattended, was swamped by an accumulation of reeds and other drift on its anchor ropes. Strenuous efforts of the whole available crew, assisted by a launch, could not free it from its sand grave in the bed of the river. It was never recovered.



(*a*) CROSS-SECTION 200 FEET ABOVE GAUGING SECTION



(*b*) CROSS-SECTION 100 FEET ABOVE GAUGING SECTION

FIG. 1.—CROSS-SECTIONS IN THE SACRAMENTO RIVER AT FREEPORT, CALIF., IN 1879.

The current meters (of the Henry type), reporting every revolution by a clicking sound, indicated a decided pulsation of the current which was attributed to the effect of the whirls in the water. The whirls were quite apparent.

"Boils" were frequent at the river's highest stages. Under the influence of an upward current, the water would rise over a circular area generally 10 to 20 ft. in diameter with a maximum superelevation of about 10 in. at the center. While no cause was ascribed to these boils at the time, the writer

concluded in later years that they must have been the result of a sudden yielding of a considerable area of river bottom.* This action leaves a cavity in the river bed into which the water plunges, is deflected upward, and becomes manifest at the surface as a "boil". On the Sacramento River the boils observed by the writer occurred at points where the depths were 20 to 30 ft. and at velocities in mid-stream of 5 to 7 ft. per sec.

In 1879, during a high-water stage, the piles of a dolphin above the Southern Pacific Railroad Bridge, at Sacramento, Calif., were floating away as if they had been inadequately driven. Soundings by the writer confirmed this view. They showed that in mid-stream the bed was then 25 ft. lower than it had been at a low-river stage a few months before.

In December, 1890, the Paine Crevasse occurred below Sacramento on the opposite side of the river. Soundings above this point revealed a tremendous scour due to accelerated current. More than 2 000 000 cu. yd. of sand had been scoured out of a 2-mile stretch of river within one day. In mid-stream, about opposite the down-stream edge of the crevasse, where there had been 20 ft. or more of water before the breach in the levee, the oars touched bottom in rowing; and a few days later there was again deep water at the same spot.

During the fall of 1878, in the mouth of the American River just above Sacramento, at the low stage of the river, a sand-bar was found along the north bank of the channel. During the winter freshet this sand-bar was swept out, and in the spring of 1879, during a protracted over-bank stage of the river, a new deposit was made, level from bank to bank, completely obliterating the old low-water channel. Through this new deposit the falling river cut a channel along the north bank a little later, and no trace of the low-water channel of 1878 along the south bank was left. It was estimated, at the time, that these river-bed changes represented a delivery of more than 340 000 cu. yd. of sand from the American into the Sacramento River, not including that delivered as an ordinary bed load.

Thus, under certain conditions, the configuration of fairly large sections of river bottom may be quickly changed. Every such change, moreover, evidences a rapid down-stream movement of sand in the river bed, probably by saltation of the material which, at low and ordinary water stages, travels in small sand waves.†

Mining Detritus.—The bed of the Sacramento River in its lower valley reaches has a sandy bottom. This section received large quantities of detrital matter from the hydraulic mines that were operated extensively along its tributary streams until about 1880. The accumulation below the mouth of the Feather River was large. This sand is clean and of a medium fineness. Originally, the water was fairly clear at low stages and never very heavily charged with sediment. Observations in 1879 indicated that the maximum quantity of material in suspension at mid-depth (still far in excess of that under original conditions) was about 0.1% by weight. The river, undoubtedly, carried much more material in suspension some years before, during the greatest activity of the hydraulic miners.

* Report of Examining Commission on Rivers and Harbors for California, 1890, p. 35.
† Described by Dr. G. K. Gilbert, in *Bulletin No. 86*, U. S. Geological Survey.

For many years the two principal tributaries of Feather River—the Yuba and Bear Rivers—were heavily charged with mining detritus. They originally entered the Great Central Valley of California in secondary valleys flanked by the higher (so-called "valley") plains. Their well-defined channels meandered through fertile bottom-lands; but the large volumes of sand dumped into the mountain reaches greatly overtaxed the transporting power of these streams. The natural channels in the secondary valleys were filled and the bottom-lands, too, were covered with a layer of clean sand. For example, in 1880 the Yuba River where it entered the main valley had obliterated its bed and had covered its bottom-lands with coarse sand 2 to 3 miles in width. It ran wild over the surface, behaving like all rivers when the supply of sand exceeds their transporting power and spreads over broad areas in shallow, unstable channels.

Why, was the first thought, had not these rivers cut deep channels and, with concentrated energy, swept the sand along instead of covering fields with it and filling houses almost to the tops of the windows? The answer is simple. The bottoms of overloaded streams are invariably flat, almost level, from bank to bank. On the other hand, streams carrying but little sand are in deep narrow channels. Whenever there is more material presented to a stream for transportation than there is capacity to transport, the stream will flatten out, increasing its width and reducing its depth. Thereby it increases the area of bottom subjected to sand traction and to scouring and correspondingly increases the volume of material pushed along as a bed load.* Many of the small streams which enter the San Joaquin Valley from the Sierra Nevada have channels of the surcharged sand type.

Progressive Bed Changes.—By the accession of sand from its principal tributaries (the Feather and the American Rivers), during the period of hydraulic mining activity, the bottom of the Sacramento River was raised progressively until about 1880. For some years thereafter the river bed near Sacramento was at a higher elevation than the water surface at low-river stages had been under natural conditions. In Fig. 2,† which shows this condition, the zero point on the gauge is 3.85 ft. above low tide in Suisun Bay, or 4.35 ft. above low tide in San Francisco Bay. From 1849 to about 1860 there was a rise and fall of the river at Sacramento due to tidal action (during the river's annual low-water period) of more than 2 ft. By 1871 the tidal range had been reduced to about 9 in., and about twenty years later had disappeared. The low-water stage at Sacramento was zero on the river gauge as late as 1856; 5.5 ft. in 1878; 7 ft. in 1880; and 8 ft. in 1889. It reached 10.5 ft. in 1890. Then bed erosion began; the material transported exceeded the supply. The low-water stage in 1894 was at 7.5 ft. on the river gauge and, by 1920, it was again at zero as in the days before hydraulic mining.

Although in the Sacramento River the low stages of recent years are due, in some measure, to the reduction of the low-water discharge by diversions

* For discussion of detrital material delivered into the rivers of the Sierra Nevada Mountains, California, by hydraulic mines from about 1853 to 1881, see "Hydraulic Mining Débris in the Sierra Nevada," by Dr. G. K. Gilbert, *Professional Paper No. 105*, U. S. Geological Survey.

† Reproduced from Rept. of Examining Commission on Rivers and Harbors of California, 1890, pp. 32–33.

for irrigation, and also to channel enlargement by dredging at down-river points for levee material, nevertheless these data indicate the passage of a great wave of débris (sand) down the river. The crest passed Sacramento about 1890 and reached the river's outfall point about 60 miles farther down stream, about 1917.

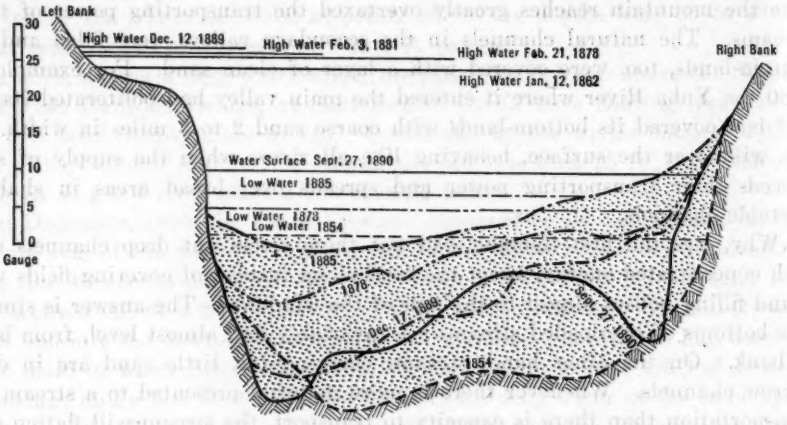


FIG. 2.—CROSS-SECTION OF THE SACRAMENTO RIVER, AT THE FOOT OF K STREET, SACRAMENTO, CALIF.

FINE SAND IN THE COLORADO

Physical Characteristics.—The rivers of the southwestern part of the United States are in a class by themselves. Of these, the Colorado is typical. At Temple Bar in 1896, about 12 miles up stream from the mouth of the Virgin River, the water looked like chocolate as it is ordinarily served. In a glass of water dipped from the surface, $\frac{1}{8}$ to $\frac{1}{4}$ in. of sediment would settle in a few minutes. A sample from the most promising mud-bar was analyzed to determine its suitability for making brick. The report showed no clay. The material was practically all silica in very fine particles.

In this part of the river there are alternate gravel-bars and deeps. In the Iceberg Canyon, a few miles farther up stream, where the bottom is a coarse gritty sand, the river is less than 250 ft. wide at its low stages and 30 to 60 ft. deep. Here and there in broad bars fine river silt is found. Some is mud and clay-like in character, shrinking and cracking as it loses moisture; some is fine sand most of which will pass through a 100-mesh sieve.

Where the river formed its great eddies the bottom was covered with this fine sediment. A loose log-boom circulated for hours in such an eddy, several hundred feet in diameter. While recapturing this boom an attempt was also made to recover a $\frac{5}{8}$-in. wire cable. Although a small scow was available and a strong pull, practically vertical, could be exerted, the cable could not be broken out of the quicksand bottom. Under water this material had a firm surface until its cohesion was broken by vibration, whereupon its behavior was momentarily that of a liquid. Over considerable areas of the bottom there were constantly occurring sudden changes of bed elevations as blocks of sand would rise and float away.

Silt Load of the Colorado River and the Imperial Canal.—The material now available for study consists of (1) determinations of the quantity of silt in suspension in Imperial Canal (which carries Colorado River water) during 1914 and 1915, by Mr. J. E. Peck; (2) determinations of bed elevation in the upper reaches of the canal, particularly from 1914 to 1920; (3) a determination of the quantity of silt in suspension in the Colorado River at Yuma, by the U. S. Reclamation Service from 1909 to date; and, finally (4), additional silt observations along the Imperial Canal and its branches, by Mr. Peck,* from June, 1917, to June, 1918.

The observations of Mr. Peck were expected to determine the amount and character of the canal's suspended load of silt. On the other hand, the soundings in the head reaches of the canal were made primarily to ascertain its constantly changing capacity to carry water. Incidentally, however, they served also in making a first estimate of the volume of material periodically deposited and eroded.

Operation of Canal.—The history of the Imperial Canal is available elsewhere.† The canal had its original head in California, at Hanlon's, about 100 yd. up stream from the International Boundary. It left the river at an oblique angle and the flow was controlled by a wooden structure. During the falling stage the head of the canal and the gate were obstructed by deposits of silt which interfered with the successful operation of the canal.

The resulting water shortage followed by failure to secure sympathetic action by Congress prompted an application to Mexico in 1904 for a permit to divert water on Mexican territory. When this privilege was granted a new connection with the river was made just below and paralleling the boundary line; but this too, was choked by silt deposits. As a further relief measure another connection was opened about 4 miles south of the International Boundary at a point where the river and canal were only about ½ mile apart. The diverted water was allowed to flow in this cut without regulating works. The result was disastrous. Erosion at an increasing rate, and, later, failure to check the increasing flow, resulted (November, 1905) in a complete deflection of the Colorado River from its original course, and the formation of the Salton Sea.‡

In the light of these facts and because silt conditions in the laterals and distributing ditches of the Imperial Canal System were giving increasing trouble, it was found desirable (1916) to provide means for silt removal from the canal at its head. Conditions both physical and political made it impracticable to construct and maintain a permanent dam in the river with a settling basin at the canal head. Therefore, suction dredges operating in the head reach of the canal were provided, one in 1918 and another in 1919. A third suction dredge was later put to work in the canal just below the Hanlon Gate. When all convenient dumping space was filled, the dredged material was

* In co-operation with C. E. Tait, U. S. Dept. of Agriculture, under the supervision of the late G. G. Anderson, M. Am. Soc. C. E., and the writer, Consulting Engineers, Imperial Irrigation District.

† *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2183; also, *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 1.

‡ "Irrigation and River Control in the Colorado River Delta," by H. T. Cory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1204.

returned to the river channel below the head-works. It was quickly swept away at the stream's next high stage.

Conditions in the Canal.—From these dredging operations in the head reach of this large canal the magnitude of the changes which must be taking place in the unstable bed of the Colorado River, can be inferred. Periodic surveys of the canal bed showed that at certain seasons there was a pronounced tendency to build up the canal bottom by silt deposits and at others a tendency to cut channel. During the years covered by the silt studies the flow of the canal ranged from a low-water stage in winter of somewhat more than 2 000 sec-ft. to a maximum at the height of the irrigation season—June to August—of between 5 000 and 6 000 cu. ft. per sec. The total annual diversion ranged from about 2 000 000 acre-ft. to more than 2 500 000 acre-ft. The sides and bottom of the canal to an unknown depth consist of the same fine silt that is carried as a bed and suspended load by the Colorado River.

Studies during 1917 and 1918, in co-operation with the U. S. Department of Agriculture disclosed that the material deposited in the canal and found in its bed and in the beds of its various branch canals, is fine sand. This is shown in Table 1. Attention is directed to the fact that the sampling of the material in the bed of the Main Canal at the Hanlon Gate on one occasion (July 15, 1917) showed in relatively coarse sand, 33% caught on a No. 40 sieve.

Alignment and Structures.—At that time the Imperial Canal partook in large measure of the character of a natural stream.* The water diverted from the Colorado River was carried for 4 or 5 miles in a dredger cut and was delivered into a channel having an irregular alignment, resulting from the enlargement by erosion of an old natural overflow channel. For miles, the canal follows this "Alamo River" and has its deeps and shoals just like a natural stream. Throughout the period covered by Mr. Peck's sampling, the water entered the canal through a cut which functioned as an arm of the river. At high-water stages, generally from March to July, partition of the water from the river to the canal intake took place just as at a point of separation of a narrow from a broad delta channel. At low-water stages the flow in the river channel was checked by a temporary weir of rock or brush. Then the canal took the major portion, and occasionally all, of the river flow.

Until 1918 this water flowed in an open cut or reach about 1 700 ft. long to the Hanlon Regulating Gate, a concrete structure with eleven openings controlled by Taintor gates. In 1918, the Rockwood Head-Gate,† about 1¼ miles above the Hanlon Gate, was completed. It is a structure of concrete with seventy-five openings controlled with flash-boards. The Rockwood Gate replaces about 600 ft. of river bank, its up-stream face being set back only a few feet from the original bank alignment.

The natural turbidity of the river at the head of the Imperial Canal, it should be noted, may have been disturbed, in some measure, by the flow and sluicing control operations at the head-works of the canal of the Yuma Reclamation Project, which are at the Laguna Dam about 20 miles up stream. Before the diverted water enters the Yuma Canal, it flows through a settling

* *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 1.

† *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2185.

TABLE 1.—CHARACTER OF SILT DEPOSITED IN THE BEDS OF CANALS OF THE IMPERIAL CANAL SYSTEM.

FIELD DETERMINATIONS BY MR. J. E. PECK.

Date.	Number of samples averaged.	Dry weight per cubic foot, in pounds.	Specific gravity.	Percentage of material caught on sieves.						
				No. 20.	No. 40.	No. 60.	No. 80.	No. 100.	No. 200.	Passing No. 200.
MAIN CANAL AT THE HANLON HEADING.										
March 14, 1917	3	.....	2.72	0	0.05	1.22	33.07	23.54	34.38	7.17
March 28, 1917	2	.....	3.64	0	0.0	0.08	5.34	17.58	59.81	17.09
May 1, 1917	6 to 12	102.	2.69	0	0.26	5.03	48.59	16.90	17.07	11.92
May 16, 1917	1	103.	.....	.....	.....	.....	.....	.....	.....	.....
July 15, 1917	5	.....	2.65	12.09	20.88	33.40	20.58	1.34	4.26	1.68
CENTRAL MAIN CANAL—NEAR 10-FOOT DROP ABOUT 52 MILES FROM HEAD.										
July to December, 1917	4.	98.	2.65	0	0.23	0.46	6.08	6.08	69.66	17.21
DAHLIA CANAL AT HEAD.										
July, 1917, to February, 1918	5 to 8	100.	2.68	0.02	0.22	0.44	0.76	1.70	64.47	32.15
DAHLIA NO. 12 CANAL AT HEAD.										
June, 1917, to July, 1918	5 to 8	90.	2.63	0.10	0.07	0.02	0.45	1.51	63.65	34.03
NO. 4 CANAL AT METER BRIDGE (HEADING).										
March, 1917, to February, 1918	7 to 9	94.	2.66	0.03	0.08	0.34	1.43	3.73	68.19	25.93
WEST SIDE CANAL AT BOUNDARY LINE.										
June 8, 1917	1	93.	2.67	0	0.40	2.00	10.10	17.55	47.60	21.90
NO. 8 CANAL AT METER BRIDGE NEAR HEADING.										
June, 1917, to February, 1918	3 to 7	99.	2.67	0	0	0.15	0.50	5.10	71.47	22.60
NO. 6 CANAL AT HEADING.										
July, 1917, to February, 1918	3 to 5	96.	2.63	0	4.62	3.72	6.92	1.97	50.70	31.67
NO. 5 CANAL AT HEADING.										
March, 1917, to February, 1918	4 to 7	99.	2.66	0	0.01	0.05	7.11	23.04	59.53	9.88
ROSITAS CANAL.										
June, 1917, to February, 1918	5 to 7	98.	2.65	0	0.08	0.22	14.04	5.54	61.56	14.12

basin so arranged that the deposited silt can be sluiced into the river below the dam. The sluice-gates are opened one day a week, when for the first few hours the river below is muddier than at other times. No correction due to this condition has been applied to any of the determinations of suspended silt in the canal by Mr. Peck, or in the river by the U. S. Reclamation Service.

Silt Loads.—The results of the observations on the canal are chiefly valuable in showing that the water supports and carries its suspended load throughout the entire length of the canal. Sometimes there is less, sometimes a little more, sediment in suspension at the head of the canal than at points 20 to 40 miles below; but, generally, when the water is very muddy at the head this condition persists through the entire length of the canal.

In 1914, the samples at the head of the canal were taken daily from the turbulent water just below the Hanlon Regulating Gate. The purpose was to get fair average samples from a single station in the head of the canal. The specific gravity of the silt in the canal averaged 2.64 and the voids in the material, as deposited in the bed, about 40 per cent.

A phenomenon illustrative of occasional conditions favoring the temporary conversion of the bed load of a river into a suspended load is the formation of a standing wave. This is noticed in streams of moderate depth, flowing in beds with fine sand bottom. It differs somewhat from the "boils" noted on the Sacramento River. It is a common phenomenon on the Lower Colorado. It represents a violent, sometimes fairly long-sustained attack on the river bed. "Sand is running", or "a sand wave is moving down stream", are phrases locally applied to this phenomenon. A standing wave is noted at the up-stream end of a disturbed area similar to that caused by a broad submerged snag in the river. Down stream the waves decrease in height, but extend over a gradually widening area and usually run out within 100 or 200 yd. Such a disturbance appears to the eye to be traveling slowly up stream, but is, in fact, fairly stationary. Undoubtedly, it is due to the yielding of a considerable stretch of the river bottom and the formation of a temporary depression which disturbs the steady flow of the stream. The waves gradually lose height and finally disappear when the depression is refilled to the original level by the river's bed load. The phenomenon is undoubtedly the result of the "anti-dune" movement of the stream's bed load which has been described by Dr. Gilbert.*

The Bed Load in the Imperial Canal

A Special Study.—A river's suspended load of silt can readily be determined by taking samples of the water at a sufficient number of points in any cross-section. The problem of ascertaining the bed load, however, is a difficult one to solve, and the engineer must usually be content with some random estimate.

It will not do to assume that, because somewhere else the bed load of a river was one-fifth, or some other fraction, of its suspended load, this ratio will hold in other streams. Such comparison is only permissible when all conditions are similar. There is no limit to the range of this relation. Near

* *Bulletin No. 86*, U. S. Geological Survey.

one extreme there are comparatively clear rivers that transport a bed load of cobbles and gravels; and at the other extreme are the streams of the Colorado type, heavily charged with fine silt flowing in beds of material differing but little from that carried in suspension. Realizing this fact, and requiring information regarding the bed load of the Imperial Canal, a special study of new and of earlier observations was made by the writer to ascertain whether changes in the elevation of the canal bottom during a number of years would "throw some light" on this subject. The stretch of canal selected extends down stream 18 000 ft. from the Hanlon Regulating Gate which, as explained, is about 1¼ miles below the Rockwood Gate in the head of the canal.

For some months after the completion of the Rockwood Gate the canal water flowed from the river through this upper gate uncontrolled, but in the summer of 1918, stop-planks were put into use and thereafter the water from the river dropped over them.

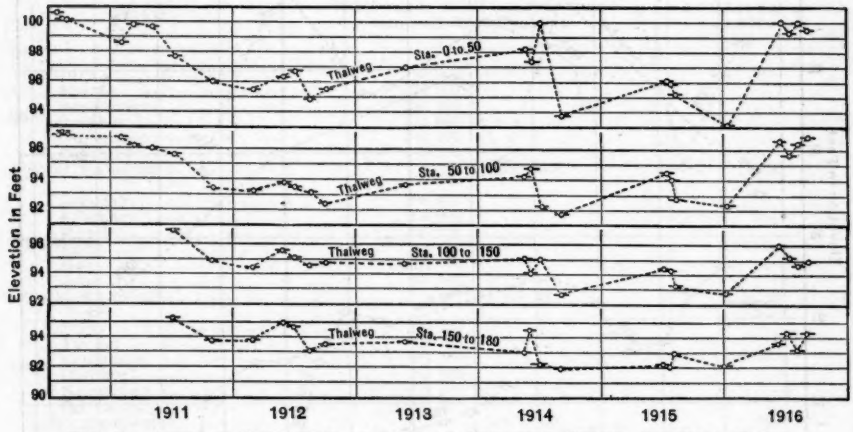


FIG. 3.—AVERAGE ELEVATION OF LINE AT GREATEST DEPTH, IMPERIAL CANAL.

Seasonal Changes.—It was found that in the spring while the river was rising from its low (winter) to a high (spring and summer) stage, more silt was deposited in the canal than its water could transport. The canal bed, at least in its upper stretches, rose to a highest position in June and was lowest in mid-winter. Considering only the selected 3.5 miles of canal, this rise, due to the deposit in the first four or five months of the year, ranged from a minimum of 1 ft. in 1912, to a maximum of 4.4 ft. in 1916. Thereupon, generally beginning in June or July, the same stretch of canal was deepened by erosion; the amount of this deepening was about 3.5 ft. in 1911, about 2.1 ft. in 1912, and about 2.7 ft. in 1915. These changes in the average elevations of the canal bed are shown diagramatically in Figs. 3 and 4. On these diagrams each point represents the average elevation of the thalweg or of three parallel lines representing the canal bottom for the particular stretch of canal as marked. These curves bring the information

relating to bottom elevation forward to the time when dredging operations in the head of the canal modified the earlier conditions.

As was expected, these studies have thrown some light on the rate of the bed-load movement of the water. It appeared obvious that an increase of deposit represented accession of silt from up stream, arriving as bed load or as material dropped from suspension, and that erosion represented depletion in excess of the material arriving from up stream and temporarily coming to rest.

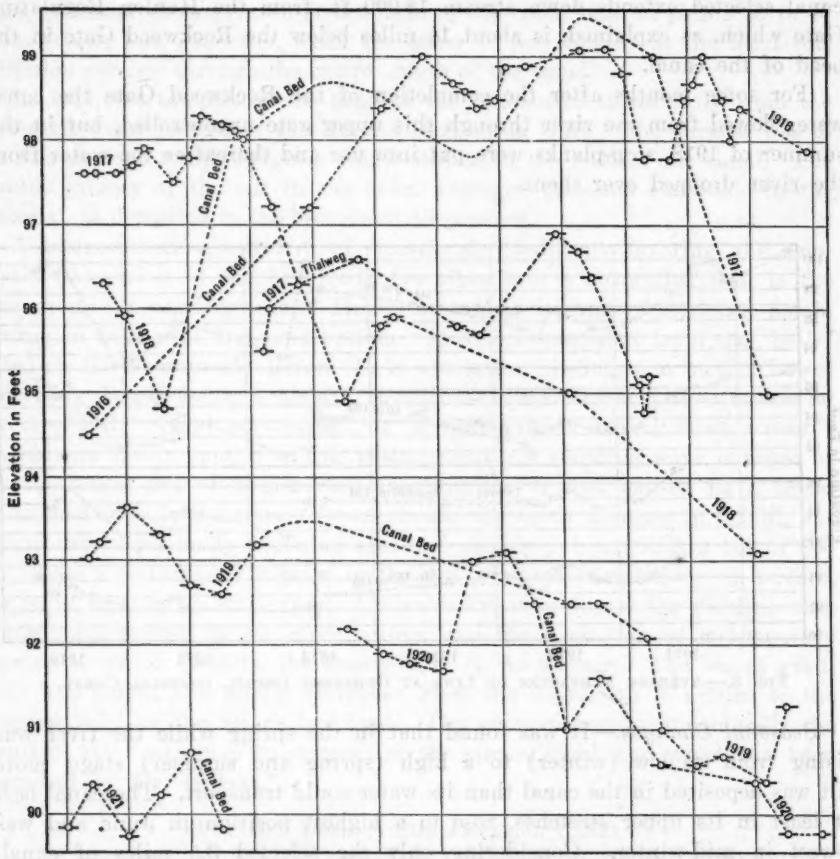


FIG. 4.—AVERAGE ELEVATION OF CANAL BED, IMPERIAL CANAL.

These studies show that during late winter and spring at least 280 000 cu. yd. of silt per month were being deposited, temporarily, in the canal stretch under observation and that a like volume was removed later in the year. How much more silt was traveling along the bed in continuous movement in sand waves could not be ascertained, but it was suspected to be large. Inspection disclosed that the canal bed, at least near shore, was covered with small sand waves about $\frac{3}{4}$ in. high and traveling slowly like sand dunes. The

individual sand particles roll up and over the crests into the troughs.* Occasionally, the water would take on a disturbed appearance for a time probably denoting temporary instability of "anti-dune" sand traction.

The annual water delivery of the canal at the time of these studies was about 2 000 000 acre-ft. The water slope was about 1.5 ft. per mile, and the mean velocity about 3 to 4 ft. per sec. The silt in suspension ranged from about 0.33% to more than 1% by weight, and averaged about 0.8 per cent.

Dredging Operations.—In 1918, the canal head-works were modified, suction dredges were installed, and dredging from the canal bed was commenced. This intercepting of the silt not only improved the regimen of the canal, but also reduced the cost of canal operation by obviating the more expensive removal of the same quantity of silt from down-stream sections and from canal laterals and ditches.

TABLE 2.—REMOVAL OF SILT BY DREDGING FROM THE HEAD-WORKS SECTION OF IMPERIAL CANAL.

Period.		Cubic yards of silt excavated and removed from the canal.		Average per month, in acre-feet.
1918	July	180 000		....
	August	224 600		....
	September	248 400		....
	October	373 000		....
	November	144 700		....
	December	298 700		....
Total, six months			1 469 400	150
1919	January	286 900		....
	February	252 900		....
	March	126 200		....
	April	181 000		....
	May	353 800		....
	June	523 500		....
Total, six months			1 724 300	170
1919	July	570 100		....
	August	629 100		....
	September	594 100		....
	October	652 800		....
	November	484 000		....
	December	756 800		....
Total, six months			3 686 900	380
1920	January	798 300		....
	February	688 700		....
	March	759 900		....
	April	952 700		....
	May	886 300		....
	June	1 134 400		....
Total, six months			5 220 300	540
1920	July	1 122 800		....
	August	479 700		....
	September	342 600		....
	October	308 200		....
	November	354 300		....
	December	283 300		....
Total, six months			2 890 900	300
Grand total and average			14 991 800	310

The silt thus removed is noted in Table 2. The estimates of volume dredged were made from samples of the discharge, except during the early

* *Professional Paper No. 86*, U. S. Geological Survey.

months of dredging when the spoil was used in filling low ground at the head of the canal to canal-bank level. On the plea of economy the amount of dredging in the head reach of the canal was reduced by the Irrigation District authorities in 1920 and was stopped with the close of that year.

Despite the large aggregate volume of dredging involved (nearly 15 000 000 cu. yd. in 30 months), the effect on the regimen of the canal considered in its entirety was hardly noticeable. General conditions remained apparently unchanged; but there was some deepening of the canal bed below Hanlon's. Certainly, the dredging was beneficial and should not have stopped. It has since been resumed for a few months in the year. While it is not quite clear what effect this dredging had on the lower reaches of the canal, it is certain that if the large quantity of the removed silt had been allowed to remain in the canal it would have traveled down stream to the annoyance of the canal management and the inconvenience of the farmer who is constantly receiving excessive quantities of silt with his muddy irrigation water.

In the upper canal reaches, even before the dredging commenced, some deepening of the canal was noted. Thus, for example, the mean elevation of the canal bottom during January and February 1918, was about 2 ft. lower than in the corresponding months of 1917. For the entire year, 1918, during the last half of which silt was removed from up-stream portions of the canal by dredging, the average lowering was also about 2 ft. In 1919, the bottom of this upper canal stretch was further deepened about 2 ft. This is to be ascribed to the depletion of the supply of silt by dredging and the continuation of bed-load movement. There was some restoration of bed elevation in 1920, the canal bottom being at about the same position in August and September as in the preceding year; and then, despite cessation of the dredging operations, the bed was lowered several feet to a position early in 1921 about 7.5 ft. lower than it had been in 1917. Some of this lowering (Figs. 3 and 4) is no doubt to be ascribed to the dredging in the head reach of the canal.

It is noteworthy that the dredging did not relieve the canal entirely of its bed load. The periodical accession and depletions of silt deposits, as evidenced by the rising and lowering of the canal bottom, continued. The dredged sumps in the head of the canal did not trap all the silt which came over the flash-boards of the Rockwood Gate. It now appears that even 500 000 cu. yd. per month (the average quantity removed for 2½ years) does not represent the entire load of fine sand carried as a bed load by the canal. How much more was being transported is not known, and no entirely satisfactory basis for estimating it has been found.

Dredging, as Table 2 shows, removed on an average about 310 acre-ft. of silt per month from the bed of the canal for 2½ years, and more than 400 acre-ft. per month for certain periods of 12 months. Had there been no removal by dredging it is reasonable to assume that this material would have been carried down the canal mainly as a bed load.

The Bed Load of River by Comparison with That of the Canal.—It appears from the foregoing that the canal, with an annual discharge of from 2 000 000 to 2 500 000 acre-ft., transported about 4 000 acre-ft. of silt (at 100

lb. per cu. ft.) by traction, as its annual bed load. There must have been, of course, some pick-up and temporary transport in suspension. Because of this fact the estimate of canal-bed load may be accepted as a minimum.

If this determination be accepted it affords means for estimating, approximately at least, the bed load of the Colorado River itself. The traction exerted by the river on its bed may be regarded as independent of the suspended load. The sand on the bottom travels by rolling or by saltation regardless of whether the water is clear or muddy. In view of general similarity of canal and river beds, the annual bed loads of the canal and of the river may be assumed to be proportional to the annual discharge. This assumption ignores the fact that the river departs most widely from the characteristics of the canal when its discharge is greatest and the attack on its bed is most vigorous. The resulting estimate, therefore, is again more likely to be too small than too large.

On this assumption the bed load of the Colorado River, with its annual discharge of about 15 000 000 acre-ft. of water, at Yuma should be about six or seven times as great as that of the canal, or 25 000 to 30 000 acre-ft. per year. The assumption as to corrresponding fineness of material applies to the material in the beds of river and canal, as well as to that in suspension. The larger velocities in the river at its high stages, if taken into account, would probably increase somewhat its relative and therefore, also, its total bed load.

TABLE 3.—SILT IN SUSPENSION IN IMPERIAL CANAL AT ITS HEAD AND IN COLORADO RIVER AT YUMA, ARIZONA, 1914.

Month.	IMPERIAL CANAL AT HANLON'S.				COLORADO RIVER AT YUMA.		
	Discharge, in second-feet.	Weight of silt, per-centage.	Volume of silt at 24 hours, per-centage.	Silt, in tons (2 000 lb.)	Discharge, in second-feet.	Weight of silt, per-centage.	Silt, in tons (2 000 lb.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
January	1 500	0.50	2.0	624	7 500	0.66	4 170
February	1 530	0.80	4.5	932	11 600	1.04	9 160
March	2 390	1.32	6.5	2 660	15 000	1.52	19 200
April	2 890	0.80	2.8	1 886	22 600	1.04	19 200
May	3 450	0.62	1.2	1 805	53 800	0.76	34 500
June	3 760	0.32	0.6	982	105 900	0.53	45 840
July	3 760	1.01	3.5	3 205	51 600	1.27	55 200
August	3 210	1.32	7.5	3 575	22 000	1.69	31 600
September	2 890	0.90	5.5	2 125	9 900	1.17	9 450
October	2 430	1.06	6.0	2 175	13 700	1.58	18 800
November	1 540	0.59	3.5	745	10 300	0.90	7 560
December	1 400	1.02	3.7	1 205	13 300	0.56	6 380
Year	2 563	0.86	....	21 927	28 100	0.93	259 860

Silt in Suspension in Imperial (Alamo) Canal.—The sampling of the water of the Imperial Canal in 1914, was done in such a way that the results indicate the canal's full load of silt (Table 3). The samples were taken from the turbulent waters just below the gates of the Hanlon Regulator where, by the flow under the gates, the bed load was converted temporarily into a

suspended load. During that year the river carried 0.93% of its weight in suspended silt. The total silt load of the canal was 0.86 per cent. These figures indicate a smaller proportion in the canal than in the river. The difference may have been considerably more, because in addition to the 0.93% in suspension, the river must have rolled considerable material along its bed.

From July, 1918, to December, 1920, while material was being removed from the canal by dredging (Table 2), the river carried in suspension 0.87% of silt. Unfortunately, there is no corresponding determination of silt in the canal. However, that which must have entered as a suspended load, computed from the volume of water and the silt content near the surface (about 0.83%), amounted to about 1 300 acre-ft. per month at 100 lb. per cu. ft. in place. Of this, 310 acre-ft. per month were removed by dredging (Table 2). The remainder was carried down the canal in part as suspended, and in part as bed load.

TABLE 4.—Silt in Suspension in Imperial Canal and in Colorado River (1917–1918).

Month.	Imperial Canal below Hanlon's.					Colorado River at Yuma.		
	Approximate canal discharge, in second-feet.	Silt, weight, percentage.	Silt per day, in 1 000 tons.*	Silt per month, in 1 000 tons.*	Mean discharge, in second-feet.	Silt, weight, percentage.	Silt per day, in 1 000 tons.*	Silt per month, in 1 000 tons.*
1917:								
June	4 500	0.360	42.2	1 270	89 900	0.54	1 305	39 150
July	4 500	0.268	32.5	1 000	93 900	0.46	1 160	36 000
August	2 300	0.377	24.4	753	23 500	0.68	432	13 400
September	4 200	0.240	27.2	815	9 000	0.30	73	2 190
October	3 600	0.385	37.3	1 158	7 600	0.29	59	1 840
November	2 300	0.113	7.1	212	7 100	0.14	27	804
December	2 000	0.080	4.4	134	6 800	0.13	24	738
1918:								
January	2 200	0.088	5.2	162	6 600	0.15	27	828
February	3 000	0.083	6.9	192	5 800	0.15	23	658
March	3 400	0.592	54.2	1 680	16 400	0.96	425	13 160
April	4 100	0.317	35.0	1 050	12 900	0.39	136	4 080
May	4 400	0.265	31.4	980	29 100	0.54	423	12 100
Weighted average	3 470	0.280	25.8	.....	23 600	0.54	344	.....
Totals	.....	.....	....	9 406	.....	....	....	125 868

* 2 000 lb.

The samplings of the canal water from June, 1917, to May, 1918 (Table 4) may be cited to show that part of what was suspended load in the river became bed load in the canal. These samples were taken below the zone of influence of the Taintor gates, below the Hanlon Regulator, where possibly some, if not all, separation of bed load from suspended load already had taken place. They showed 0.28% of silt in suspension, while in the same period the river water carried 0.54 per cent. Based on the mean flow of the

canal (3 470 sec-ft.) and the assumption that the water which flowed unobstructed from the river into the canal was charged with the same quantity of silt as at Yuma, the total volume of silt which entered the canal was about 700 cu. yd. per month, of which 340 cu. yd. became a bed load or temporarily came to rest in the canal bed.

It should be noted, however, that at 0.54% the river water was materially short of the silt contents determined at Yuma, 0.93% for the longer period of 30 months, of which only the first 6 months are covered by simultaneous sampling.

The sampling of the disturbed or turbulent water of the canal just below the Taintor gates at Hanlon's in 1914, indicated a transport of silt by the canal water as shown in Table 3. The volume of wet silt "after 24 hours" (Column (4)), shows the variable rate at which the silt in suspension settles to the bottom of a container. The data are not intended, however, to indicate the space it will occupy when finally incorporated in the geologic structure of the river's delta region. This volume can be computed from the unit weight of the deposited material. For convenience a weight of 100 lb. per cu. ft., about the weight of the material in the beds of the main canals, has been assumed (see Table 1). As small a value as 85 lb. per cu. ft. is sometimes assumed in converting weight to volume, particularly in estimating the space that the silt which is deposited under water will occupy in a reservoir. The studies made on the Imperial Canal and its branches indicate that the larger weight is more nearly correct for this locality.

The observations in the canal, listed in Table 4, were made some distance below the Hanlon Gate and results were reported by Messrs. C. E. Tait and J. E. Peck, the sampling having been done by Mr. Peck. The observations in the Colorado River are those of the U. S. Reclamation Service. Where figures do not agree with original records it is due to the fact that it was necessary to correct averages of incomplete records by approximating missing observations. Furthermore, the values noted in this table are weighted means and not averages. The figures presented in Table 3 depart in some instances widely from those computed by the U. S. Bureau of Reclamation. This is due to the fact that the Bureau has determined monthly values as straight averages of the daily percentage determinations, while, in preparing the tables in this paper, except as otherwise noted, each daily observation was duly weighted according to the discharge to which it applies.

Variation in Silt Content.—Comparison of values in Table 3 shows that during the months of May and June, 1914, the volume of silt in suspension expressed in weight (Column (3)) was only one-half as great as the volume that settled in the bottom of a tube in 24 hours (Column (4)). In other months, February, March, September, and October, 1914, the percentage by volume exceeded that by weight sixfold. These large variations suggest lack of uniformity in the character and condition of the river's suspended load, at first attributed to the occasional large admixture of Gila River water, just above Yuma. This seemed to be borne out by the relative river stages in 1914; but, later, daily observations (June, 1917, to June, 1918, inclusive) failed to substantiate this conclusion. However, when the water was heavily

charged with silt, the settling nearly always was slower than when its load was light.

The canal does not hold quite as much silt in suspension as the river. This is ascribed to the fact that it is less turbulent than the river and, in consequence, the heavier grains of silt drop to its bed, thus becoming a part of its bed load. The difference in the suspended loads of the canal and the river was most striking in 1917 and 1918 (see Table 4), the figures being: For the canal, 0.28% and, for the river, 0.54 per cent.

Sampling of canal water was carried on systematically throughout the canal system from June, 1917, to May, 1918, the water being followed through the main canal into its many branches. The records are too voluminous to be reproduced herein in detail. The monthly summaries, however, have been assembled in Table 5.

It is to be regretted that during this period the Colorado River was never very muddy for longer than a few days. Nevertheless, Table 5 shows but slight change in the canal's suspended load of silt with distance from the intake. The suspended silt remains in suspension; the only exceptions are in the West Side laterals in which the smaller silt burden is explained by the opportunity afforded in its feeder, the Solfatara Canal, to drop a part of its load in a basin, provided for the purpose, along the northern base of Volcano Lake Levee.

Silt in Suspension in Colorado River at Yuma, Arizona

Sampling of River Water at Yuma.—Since 1909 the U. S. Geological Survey, in co-operation with the U. S. Reclamation Service, has made observations on the Colorado River at Yuma to determine the volume of silt transported. Twice a week samples have been taken in three verticals of a cross-section (one at mid-stream and the others at about the quarter-points). They were taken at the water surface, at mid-depth, and near the bottom of each vertical.

The results from 1909 to 1918 are presented in Table 6. The average percentage of silt carried as a suspended load during the entire ten years (0.89%) was found by comparing the total discharge with the aggregate weight of the material carried in suspension, as determined month by month. The percentages reported by the U. S. Reclamation Service for each month were accepted (for this period only) in making the calculations, although it is understood that they represent straight monthly averages instead of weighted averages. The error thus introduced for this particular period, although it is worth considering for individual months, is not material for present purposes. In Table 6, the volume of silt is computed at 100 lb. per cu. ft. The percentage of silt is as reported by the U. S. Reclamation Service. The individual determinations (8 or 9 months) appear to have been averaged—not weighted—and the monthly averages are, therefore, occasionally subject to material correction.

Distribution Throughout Year.—The large volume of silt considered in relation to discharge in January, February, and March (Table 6), is not

TABLE 5.—SILT IN SUSPENSION IN THE IMPERIAL (ALAMO) CANAL SYSTEM, 1917-1918.*

Miles below canal intake.	Location.	Percentage by Weight.											
		1917.							1918.				
		June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May
0.5	Main Canal, near Heading	0.36	0.27	0.58	0.24†	0.39	0.11	0.08	0.09	0.08	0.59	0.32	0.27
27.0	Main Canal, at Cudahy Check	....	....	0.68	0.41	0.41	0.13	0.15	0.13	0.14	0.66	0.41	0.41
35.0	Main Canal, at Alamo Mocho	....	....	0.62	0.28	0.38	0.15	0.12	0.11	0.12	0.59	0.38	0.37
41.0	Main Canal, at Lawrence Heading	0.40	....	....	....	0.36	0.19	0.24	0.12	0.12	0.53	0.36	0.29
48.0	East Side Canal, at Check 1	0.31	0.18	0.61	0.35	0.32	0.11	0.12	0.14	0.15	0.70	0.36	0.32
71.0	East Side Canal, at Myrtle Check	0.37	0.18	0.68	0.34	0.37	0.15	0.10	0.14	0.16	0.65	0.38	0.32
86.0	East Side Canal, at Junction	0.39	0.24	0.60	0.29†	0.33	0.12	0.09	0.13	....	0.55	0.23	0.34
52.0	Central Main, at 10-Ft. Drop	0.41	0.27	....	0.34	0.42	0.15	0.08	0.10	0.13	0.63	....	0.31
59.0	Dahlia Canal, at Heading	0.57	0.32	0.67	0.43	0.43	0.14	0.12	0.12	0.15	....	0.55	0.49
69.0	Dahlia Canal, at No. 12	0.48	0.28	0.62	0.35	0.40	0.11	0.10	0.09	0.08	0.47	0.34	0.27
76.0	Brawley Canal, at No. 4	0.53	0.27	0.66	....	0.28	0.14	0.07	0.10	0.11	0.62	0.43	0.34
56.0	West Side Canal, at Wisteria	....	....	0.12	0.17	0.11	0.03	0.08	0.13	0.04	0.20	0.34	0.25
65.0	West Side Canal, at Boundary	0.30	0.18	0.12	0.19	0.16	0.05	0.05	0.08	0.06	0.20	0.29	0.21
91.0	West Side Canal, at No. 8	0.38	0.23	0.18	....	0.24	0.16	0.10	0.09	0.15	0.21	0.26	0.15
104.0	Tripolium Canal, at Heading	0.31	0.28	....	....	0.23	0.10	0.07	0.08	0.12	0.23	0.30	0.28
53.0	Imp. Water Co. No. 5, at Heading	0.45	0.26	0.57	0.30	0.31	0.11	0.09	0.11	0.14	0.87	0.37	0.30
60.0	Rositas Canal, at Heading	0.53	0.30	0.52	0.34	0.57	0.23	0.22	0.19	0.19	0.68	0.41	0.48
83.0	North End Canal, at Heading	0.60	0.36	0.56	0.40	0.85	....	....	....	0.27	0.39	0.47	0.60

* The values presented in this table are the averages of daily determinations. The field work was in charge of Mr. J. E. Peck.

† Approximations from incomplete records.

fairly representative of ordinary winter conditions for the reason that in one year of the period (1916) the winter discharge of the river at Yuma was multiplied five or six times by high stages of the Gila and other lower tributaries. The admixture of these unusual freshet waters increased the average volume of silt in suspension in January, 1916, to 3% by weight, whereas ordinarily it is between 0.2% and 1.0 per cent. The discharges in February and March, to a lesser degree, were also affected. In February, 1915, the Colorado, with more than twice its ordinary discharge, had very muddy water, the silt averaging 2.48 per cent. The data for the winter months of this particular group of years, because of such extreme conditions, do not properly indicate the probable or normal relation between discharge and silt in suspension.

In the nine years, 1919 to 1927 (Table 7), all original semi-weekly observations were given weight according to river discharge in the computation of monthly mean percentages. Consequently, these are to be regarded as more nearly in accord with actual fact than those preceding 1919, particularly for the individual months.

The total quantity of silt annually transported in suspension past Yuma, estimated from these dependable figures, is shown in Table 7; also, the average volume and the probable volume of silt per calendar month for the 9-year period.

For convenience of reference the amount of silt in suspension in the Colorado River at Yuma is given in Table 8 by years. A summary of data given in Tables 6 and 7 is listed in Table 8, together with data concerning the columns of silt in suspension. The percentages from 1911 to 1918 are based on observations which were not weighted to get monthly means. The figures for these years are, therefore, subject to some correction. The percentages from 1919 to 1927 are true means and are, therefore, to be accepted as thoroughly dependable.

Comparative Amount of Silt—Surface to Bottom

Conditions Affecting Data.—The long-continued observations at Yuma are particularly valuable, not only because they furnish a dependable basis for estimating the quantity of silt in suspension, but also because they throw light on the distribution of the silt throughout a body of flowing water. The width of the Colorado River at the sampling station is about 650 ft. The verticals in which samples were taken were generally 130, 325, and 520 ft. from the left bank.

Occasionally, samples from near the bottom show excessive quantities of silt when compared with the corresponding samples at mid-depth and near the surface. These no doubt reflect a momentary condition of bed-load saltation and should be connected with the river's bed load rather than with its suspended load. In the following deductions such clearly discordant determinations of silt in suspension have been disregarded.

The Yuma Reclamation Project Canal taps the river at Laguna Dam about 12 miles up stream from Yuma. The head-works of the canal at this point include a desilting arrangement already described. When the tem-

TABLE 6.—SILT CARRIED IN SUSPENSION BY COLORADO RIVER AT YUMA, ARIZONA, BY MONTHS, 1909–1918.

Year.	Designation.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1909	Discharge, in thousands of acre-feet	.....	.....	.....	.....	3 324	6 240	4 897	2 509	2 889	861	.....	.....
	Silt, percentage	.....	.....	.....	.....	0.57	0.43	0.94	0.95	1.13	0.37	.....	.....
	Silt, in acre-feet	.....	.....	.....	.....	11 840	16 750	28 750	14 850	20 400	1 980	.....	.....
1910	Discharge, in thousands of acre-feet	.....	.....	.....	1 910	3 473	2 798	904	691	367	.....	467	427
	Silt, percentage	.....	.....	.....	0.57	0.58	0.55	0.40	0.24	0.61	.....	0.36	0.28
	Silt, in acre-feet	.....	.....	.....	6 100	12 600	9 630	2 260	652	1 390	.....	1 050	745
1911	Discharge, in thousands of acre-feet	541	743	1 068	1 214	2 765	8 819	3 094	1 182	530	1 757	722	465
	Silt, percentage	1.81	1.09	1.06	0.84	0.86	0.74	1.69	1.27	0.51	2.33	0.51	0.20
	Silt, in acre-feet	2 770	5 060	7 060	850	14 870	17 600	32 250	9 000	2 680	25 600	2 300	580
1912	Discharge, in thousands of acre-feet	331	424	818	254	2 609	6 397	2 867	1 397	582	677	699	403
	Silt, percentage	0.16	0.23	0.79	.45	0.96	0.52	0.71	0.78	0.84	0.46	1.16	0.24
	Silt, in acre-feet	331	610	4 040	530	15 060	20 750	12 750	6 800	3 060	1 945	5 080	605
1913	Discharge, in thousands of acre-feet	238	337	558	1 524	2 880	2 827	1 308	580	525	635	472	398
	Silt, percentage	0.13	0.13	0.31	1.15	0.73	0.65	0.41	0.61	1.69	1.66	0.25	0.29
	Silt, in acre-feet	230	274	1 078	10 950	10 840	11 500	3 340	2 210	5 550	6 840	1 030	956
1914	Discharge, in thousands of acre-feet	462	646	923	1 384	3 308	6 575	3 168	1 350	591	840	611	818
	Silt, percentage	0.66	1.04	1.52	1.04	0.76	0.53	1.27	1.69	1.17	1.68	0.90	0.56
	Silt, in acre-feet	1 910	4 200	8 760	8 870	15 700	21 750	25 200	14 150	4 330	8 270	3 440	2 860
1915	Discharge, in thousands of acre-feet	568	1 505	951	1 789	2 941	2 892	1 897	685	270	442	356	354
	Silt, percentage	0.128	2.48	0.94	1.24	1.37	0.82	0.64	1.52	0.57	0.70	0.47	0.41
	Silt, in acre-feet	4 500	23 400	5 580	13 880	25 250	14 800	7 580	6 510	964	1 940	1 042	906
1916	Discharge, in thousands of acre-feet	2 637	1 615	2 201	2 118	3 363	3 539	2 258	1 675	736	1 636	707	454
	Silt, percentage	2.99	1.78	1.71	1.20	0.93	0.66	0.68	1.95	0.57	1.91	0.68	0.32
	Silt, in acre-feet	49 100	17 400	28 550	15 850	19 700	20 800	9 600	20 400	4 000	19 700	3 000	906
1917	Discharge, in thousands of acre-feet	562	440	603	1 565	3 030	5 350	5 775	1 442	536	465	422	420
	Silt, percentage	0.48	0.31	0.49	1.08	0.98	0.54	0.46	0.68	0.30	0.29	0.14	0.13
	Silt, in acre-feet	1 680	863	1 845	10 050	18 550	19 150	16 550	6 140	1 005	844	370	338
1918	Discharge, in thousands of acre-feet	405	323	1 008	767	1 787	3 675	2 660	710	406	474	480	451
	Silt, percentage	0.15	0.15	0.96	0.39	0.54	0.53	1.01	0.68	0.63	0.48	0.24	0.23
	Silt, in acre-feet	380	302	6 060	1 865	6 030	12 400	16 750	3 020	1 600	1 420	1 010	650
Average, 8 to 10 years	Discharge, in thousands of acre-feet	717	754	1 016	1 478	2 888	4 411	2 881	1 207	743	865	548	465
	Silt, percentage	1.70	1.39	1.14	0.98	0.83	0.59	0.86	1.11	0.97	1.41	0.59	0.33
	Silt, in acre-feet	7 600	6 520	7 240	8 600	15 000	16 350	15 520	8 340	4 490	7 590	2 035	950

TABLE 7.—SILT CARRIED IN SUSPENSION BY COLORADO RIVER AT YUMA, ARIZONA, BY MONTHS, 1919-1927.

(Volume of silt computed at 100 lb. per cu. ft. All results of daily observations have been weighted.)

Year.	Designation.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1919	Discharge, in thousands of acre-feet	231	398	544	1 230	2 220	2 040	1 240	658	307	325	607	947
	Silt, percentage	0.16	0.34	0.86	0.97	0.50	0.55	2.04	2.54	1.10	7.78	0.54	1.59
	Silt, in acre-feet	231	846	2 920	7 460	6 940	7 012	12 810	10 460	2 110	1 584	2 049	9 410
1920	Discharge, in thousands of acre-feet	701	2 190	1 110	1 210	2 840	7 680	2 650	1 090	502	400	619	452
	Silt, percentage	0.54	1.74	1.68	1.08	0.88	0.62	0.69	0.87	(0.76)	0.24	0.40	0.21
	Silt, in acre-feet	2 366	23 800	11 660	8 166	15 620	30 460	11 430	5 926	(2 384)	600	1 548	598
1921	Discharge, in thousands of acre-feet	427	409	824	815	2 670	6 600	2 820	2 160	1 070	552	448	633
	Silt, percentage	0.24	0.18	0.58	0.34	0.76	0.50	0.74	2.34	2.32	0.45	0.33	0.79
	Silt, in acre-feet	640	460	2 986	1 781	12 680	20 620	13 040	31 600	15 520	1 552	924	3 124
1922	Discharge, in thousands of acre-feet	799	600	996	1 140	3 440	5 810	1 940	744	524	240	326	455
	Silt, percentage	0.93	0.63	1.51	1.08	1.10	0.59	0.57	1.19	1.19	0.17	0.22	0.27
	Silt, in acre-feet	4 646	2 361	9 404	7 695	23 640	21 410	6 910	5 535	3 898	255	448	768
1923	Discharge, in thousands of acre-feet	328	321	540	1 080	2 880	5 070	2 620	1 510	1 270	736	828	666
	Silt, percentage	0.19	0.18	0.37	0.86	0.72	0.61	0.79	2.00	3.17	1.67	1.41	0.85
	Silt, in acre-feet	390	361	1 249	5 806	12 950	19 330	12 930	18 870	25 160	7 690	7 298	3 536
1924	Discharge, in thousands of acre-feet	785	481	520	1 316	2 568	3 195	1 123	288	184	244	369	355
	Silt, percentage	1.45	0.45	0.44	1.39	0.86	0.76	0.62	0.66	0.75	0.71	0.34	0.25
	Silt, in acre-feet	7 112	1 352	1 430	11 430	13 780	15 170	4 353	1 188	862	1 082	784	544
1925	Discharge, in thousands of acre-feet	212	349	531	1 090	1 780	2 498	1 740	738	1 210	1 150	677	474
	Silt, percentage	0.32	0.33	0.33	0.64	0.53	0.67	0.76	0.85	2.90	1.84	0.65	0.46
	Silt, in acre-feet	424	728	1 095	4 360	5 896	10 460	8 267	3 922	21 930	13 230	2 750	1 362
1926	Discharge, in thousands of acre-feet	356	288	446	1 410	2 776	3 560	1 417	536	260	488	255	459
	Silt, percentage	0.30	0.26	0.50	1.00	0.83	0.65	0.83	0.53	0.50	2.09	0.30	0.55
	Silt, in acre-feet	667	468	1 393	8 817	14 390	14 460	7 350	1 775	813	5 724	478	1 578
1927	Discharge, in thousands of acre-feet	280	1 070	719	959	2 990	3 450	2 670	913	1 800	1 050	732	462
	Silt, percentage	0.25	1.27	0.77	0.85	0.81	0.63	0.94	1.08	3.04	1.08	0.62	0.34
	Silt, in acre-feet	437	8 490	3 460	5 095	15 130	13 580	15 690	6 165	34 200	2 089	2 837	982
Average, 9 years	Discharge, in thousands of acre-feet	458	678	692	1 139	2 684	4 434	2 024	960	792	570	540	545
	Silt, percentage	0.66	1.03	0.91	0.95	0.80	0.61	0.84	1.58	2.40	1.21	0.63	0.71
	Silt, in acre-feet	1 879	4 317	3 955	6 729	13 498	13 447	10 642	9 493	11 875	4 312	2 124	2 434

porary discharge through the Laguna Dam sluice-gates is large in relation to the entire flow—as at low stages—the picture of silt conditions may not be dependable. At high stages this disturbing cause is negligible.

TABLE 8.—SUMMARY OF SILT CARRIED IN SUSPENSION BY COLORADO RIVER AT YUMA, ARIZONA.

Year.	Discharge, in acre-feet.	Silt in Suspension.	
		Percentage by weight.	At 100 lb. per cu. ft., in acre-feet.
1911	17 840 000	1.13	126 120
1912	18 358 000	0.65	74 561
1913	11 772 000	0.74	54 798
1914	20 656 000	0.93	119 440
1915	14 645 000	1.16	106 352
1916	22 939 000	1.42	204 006
1917	20 610 000	0.59	76 385
1918	13 146 000	0.63	51 487
1919	10 747 000	1.00	66 832
1920	21 444 000	0.86	114 553
1921	19 428 000	0.86	104 877
1922	17 014 000	0.82	86 970
1923	17 849 000	1.04	115 570
1924	11 424 000	0.83	59 097
1925	12 449 000	0.96	74 418
1926	12 201 000	0.76	57 913
1927	17 095 000	1.06	113 155
Means	16 450 000	0.92	94 500

Classification of Findings.—In 1916, after seven years of observations in more than 1 100 verticals (three samples each), studies of the results of these observations were made by the writer to confirm the conclusion that the admission of river water to the Imperial Canal over flash-boards, at a proposed new head-gate, would reduce the entrained bed load of silt, and, perhaps, in some measure, also, the suspended load, when compared with flow under gates of the ordinary type. The sampling at Yuma has continued to the present by the U. S. Bureau of Reclamation, for the U. S. Geological Survey and, therefore, it has been possible to supplement and extend the studies of 1916.

The observations from 1909 to 1916 were first separated into groups, according as the silt content (by weight) was: (1), less than 1%; (2) between 1% and 2%; and (3) exceeded 2 per cent. There were 769 verticals in Group (1); 315 in Group (2); and 81 in Group (3).

A combination of the results in all verticals (1 165 in number), with sole regard to the relative quantity of material in suspension near the water surface, at mid-depth, and near the bottom, was then made. Similar combinations were made, too, for each calendar year.

The results of the studies undertaken in 1916 are shown in Table 9. The results of the more recent studies covering the period from 1919 to 1927, are given in Table 10. These data show that a little more material in suspension is to be expected at mid-depth than at the surface and still more near the bottom. In preparing Table 10, the observations were grouped roughly by periods during which the river's silt load remained fairly uniform.

TABLE 9.—Distribution of the Silt in Suspension from Water Surface to Bottom in Colorado River at Yuma, Arizona.

(From observations by U. S. Reclamation Service, 1909 to 1916.)

Year.	Number of verticals.	Silt in Percentage by Weight.		
		At surface.	At mid-depth.	At bottom.
1909	24	0.486	0.594	0.794
1910	66	0.452	0.526	0.571
1911	156	1.040	1.063	1.095
1912	201	0.652	0.660	0.682
1913	195	0.835	0.845	0.889
1914	272	0.974	1.011	1.126
1915	196	0.901	0.922	0.998
1916	55	1.280	1.441	1.485
Weighted average	1 165	0.86	0.89	0.96
Light load	769	0.49	0.51	0.59
Moderate load	315	1.29	1.33	1.40
Heavy load	81	2.67	2.74	2.78

Turbidity Varies with Depth.—It is to be noted that in the Colorado River, which is a very muddy stream, the increase of the suspended load downward from the surface bears no fixed relation to the loads at the surface or at mid-depth. The increase with depth is very nearly constant in the sense of being independent of the total suspended load. Thus, for example, the 769 separate determinations from 1909 to 1916, during periods with a load of about 0.5%, show an actual increase from 0.49% at the surface to 0.59% at the bottom, a percentage difference of 0.10. The 315 determinations in the same years, during periods with a load of 1.0 to 1.5%, showed 1.29% at the surface and 1.40% at the bottom, a percentage increase of 0.11; and the 81 determinations, when the load of silt was heavy, showed 2.67% at the surface and 2.78% at the bottom, or an increase in percentage of 0.11 at full depth. The combination of results in all the 1 165 verticals indicates that the percentage of silt at mid-depth is greater than that at the surface by 0.03 of the weight of water; and that at the bottom the percentage of silt exceeds that at the surface by 0.10. Corresponding comparisons for the period, 1919 to 1927, show the following increases from surface to bottom: For a light silt load, 0.12% of the weight of the water; for a moderate load, 0.08%; and for a heavy load, 0.15 per cent. The combination of results in all the 1 841 verticals of this period show 0.73% silt at the surface; 0.77% at mid-depth; and 0.84% at the bottom, the increase of the percentage being 0.04 from surface to mid-depth, and 0.11 from surface to bottom. In other words, if the percentage by weight of the suspended material in the Colorado River at the surface is represented by p, the percentage of suspended material at mid-depth may be taken at $p + 0.04$ and that at the bottom at $p + 0.10$.

TABLE 10.—DISTRIBUTION OF THE SILT IN SUSPENSION FROM WATER SURFACE TO BOTTOM IN COLORADO RIVER AT YUMA, ARIZONA.

(From observations by U. S. Geological Survey, 1919 to 1927.)

Year.	Number of days.	Number of verticals.	Average depth, in feet.	Silt in Percentage by Weight.		
				At surface.	At mid-depth.	At bottom.
Light Load of Silt in Suspension.						
1919	24	44	7.6	0.274	0.274	0.362
1920	51	122	7.3	0.397	0.444	0.502
1921	42	103	8.0	0.316	0.328	0.409
1922	45	60	8.8	0.311	0.311	0.409
1923	38	57	9.3	0.253	0.262	0.346
1924	39	49	8.4	0.248	0.245	0.346
1925	44	85	8.4	0.327	0.415	0.538
1926	57	90	8.0	0.304	0.337	0.466
1927	9	15	6.4	0.170	0.194	0.313
Average weighted.....	349	625	8.2	0.306	0.332	0.429
Moderate Load of Silt in Suspension.						
1919	46	101	9.7	0.486	0.524	0.587
1920	14	29	21.6	0.720	0.670	0.694
1921	40	65	13.0	0.534	0.556	0.593
1922	31	89	14.6	0.634	0.663	0.730
1923	41	110	13.0	0.711	0.705	0.712
1924	46	90	11.0	0.700	0.668	0.733
1925	17	35	15.0	0.598	0.550	0.590
1926	34	82	13.0	0.740	0.778	0.974
1927	71	150	13.0	0.707	0.823	0.883
Average weighted.....	340	759	13.1	0.603	0.629	0.681
Heavy Load of Silt in Suspension.						
1919	27	73	8.5	1.91	1.99	2.07
1920	20	60	11.6	1.42	1.45	1.39
1921	15	43	10.6	2.22	2.34	2.22
1922	27	62	11.3	1.24	1.36	1.55
1923	22	53	9.2	2.36	2.29	2.39
1924	13	37	9.9	1.40	1.44	1.42
1925	18	50	13.1	2.22	2.28	2.21
1926	8	24	7.6	2.31	2.43	2.50
1927	19	55	11.4	1.77	2.02	2.18
Average weighted.....	169	457	10.4	1.83	1.93	1.96
1919–1927, total and means....	858	1 841	10.6	0.73	0.77	0.84

Quantity of Silt in Relation to River Stage.—The Colorado River at Yuma flows on a bed of very fine sand. The three-point determinations of silt in suspension, of which use was made in the foregoing comparisons, did not include observations at points were the depth was less than 5 ft. The average depths at the included sampling stations are indicated in Table 10. The mean velocity of the river at low stages is generally 2 to 4 ft. At high stages it may go to 6 or 7 ft. per sec. When the river is in equilibrium, there

is as much sand being picked up from the bed as is dropped from the suspended load. At such times the river bed, underfoot, appears firm, smooth, and comparatively hard, like a felt-covered floor. With a current of about 3 to 4 ft. per sec., attack on the bed quickly begins whenever there is local acceleration of velocity such as may be caused by even small obstructions, for example, the legs and feet of a man standing in the stream.

Expressed in relation to the weight of the water, there is a wide range in the turbidity of the river, as the long continued observations disclose. The tabulated results show that the Colorado River is not as muddy at low stages as at moderately high stages—stages with a discharge of 15 000 to 50 000 sec-ft. They also show that the turbidity, expressed in percentages, decreases as the river goes to still higher stages. This does not mean, however, that the river's load of silt decreases with increasing volume. This will become apparent by reference to Fig. 5, which shows the relation of the volume of silt carried in suspension, expressed in acre-feet per month (estimated at 100 lb. per cu. ft.), to the river discharge, this being noted both in acre-feet per month and in second-feet. In plotting Fig. 5, only the mean monthly values as determined for the two periods, 1909 to 1918 and 1919 to 1927 (see Tables 6 and 7), were used. The curve representing volume of silt was first plotted and the percentage curve was deduced therefrom.

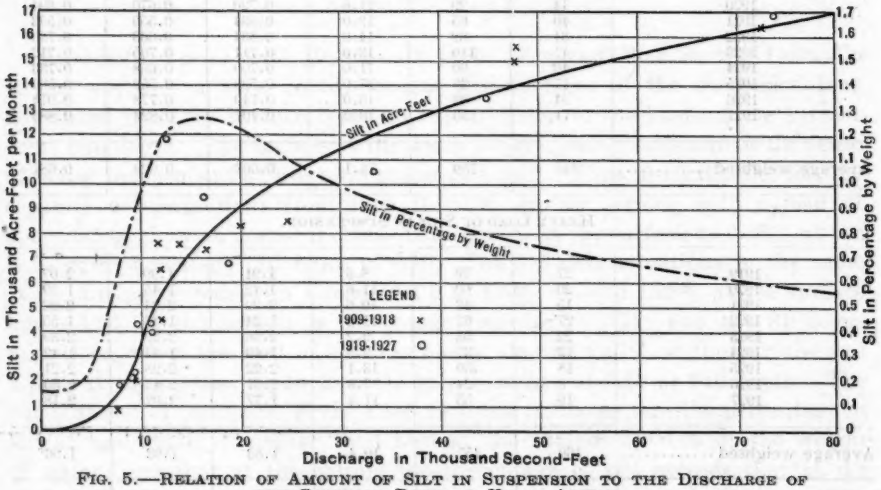


FIG. 5.—RELATION OF AMOUNT OF SILT IN SUSPENSION TO THE DISCHARGE OF THE COLORADO RIVER AT YUMA, ARIZONA.

This study should possibly have been supplemented by a more minute examination of results at the river's lowest stages when the discharge is 6 000 sec-ft., or less, but in view of the complication resulting from the desilting operations at Laguna Dam where upward of 1 000 sec-ft. of desilted water are frequently under diversion into the Yuma Canal, this additional study was not attempted.

The curves in Fig. 5 show a moderate increase of load until the river reaches a 6 000 sec-ft. stage, thereupon the load in suspension, considered in

its entirety, rapidly increases from about 1 000 to 10 000 acre-ft. per month, this quantity being reached when the river is at a 25 000 sec-ft. stage. Thereafter, the increase of load continues as the river rises, but the rate of increase is less rapid. At 80 000 sec-ft. the silt carried in suspension amounts to about 16 500 acre-ft. per month, and, for still higher stages, there is an increase of about 700 acre-ft. for each additional 20 000 sec-ft. of discharge. The curve denoting percentage of silt shows that the Colorado River is muddiest when it is discharging 15 000 to 30 000 sec-ft.

The probable volume of silt carried in suspension by the Colorado River changes not only with the discharge of the river, but also from month to month through the year. This change occurs with some regularity, as is made clear by the curves in Fig. 6. In preparing the curves of probable volume the average values ascertained for calendar months for the two periods, 1909-1918 and 1919-1927, were used. Turbidity has two periods of maxima in the year, but with frequent wide departure from mean values. The probable turbidity at any time of the year appears in Fig. 6.

Fig. 6 indicates a total annual load of silt in suspension in the Colorado River of about 94 000 acre-ft., estimated at 100 lb. per cu. ft. (which is equivalent to 111 000 acre-ft. at 85 lb. per cu. ft. of deposited silt). This does not include the river's bed load, which, as shown elsewhere, may amount to about 20 000 to 30 000 acre-ft. per annum. Considered from the standpoint of probability and by months it is to be noted that silt transportation is at a maximum in June at the river's spring high stage. It is at a minimum in the winter months when the river is low. The turbidity is highest in the early spring and in the fall. It is lowest in June at the river's highest stage and in the winter at its lowest stages. The silt carried in suspension since the beginning of the observations by the U. S. Reclamation Service, has amounted to 0.90% by weight of the river's total discharge.

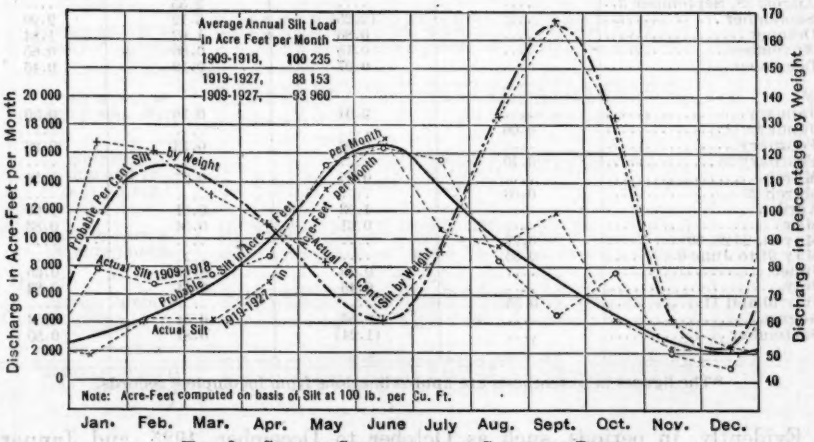

Average Annual Silt Load in Acre Feet per Month
1909-1918, 100 235
1919-1927, 88 153
1909-1927, 93 960
Discharge in Acre-Feet per Month
Discharge in Percentage by Weight
Probable Per Cent Silt by Weight
Probable Silt in Acre-Feet per Month
Actual Silt 1909-1918
Actual Silt 1919-1927
Actual Silt
Actual per Cent Silt by Weight
Actual Silt in Acre-Feet per Month
Note: Acre-Feet computed on basis of Silt at 100 lb. per Cu. Ft.
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 6.—SILT IN SUSPENSION IN COLORADO RIVER.

Perhaps there are other causes affecting the degree of muddiness besides the river stage. It is suggested by Mr. J. F. Breazeale that the silt in the

lower river in many ways shows "signs of being dispersed, that is, the colloidal particles seem to be carrying negative charges."* He believes that the source of the water and its chemical composition affects the degree of dispersion, or of flocculency, as the case may be, and, therefore, causes a greater or smaller volume of silt in suspension. He explains that when clays are flocculated, the individual particles are fairly large, but when they are dispersed, carrying charges of negative electricity, each flocculated particle splits into many subdivisions, sometimes so small as to be colloidal.

Variation of Suspended Silt Along the Colorado River

Since 1925 the U. S. Geological Survey has made determinations of the quantity of material in suspension in the waters of the Colorado River at a number of points up stream from Yuma. Some of the results are accessible in published form.† These data have enabled a preliminary comparison of the quantities of silt in suspension, expressed in percentage of weight, to be made at Lee's Ferry, Grand Canyon, Topock, and Yuma, as shown in Table 11. These values are based, at least in part, on composite samples.

TABLE 11.—Silt in Suspension at Various Points of the Colorado River.*
(From U. S. Geological Survey records.)

Period.	Percentage of Silt in Suspension by Weight.			
	Lee's Ferry.	Grand Canyon.	Topock.	Yuma.
1925:				
August	....	....	....	0.85
August 14-20	....	....	0.79	....
August 22-27	....	....	1.02	....
August 23-29	....	2.54	....	....
August 28, September 3	....	....	2.55	....
September	....	(2.25)	3.12	2.90
October	....	0.86	1.62	1.84
November	....	0.15	0.36	0.65
December	....	0.07	0.19	0.46
1926:				
January	....	0.04	0.16	0.30
January 11	0.06	....	....	....
February	....	0.08	0.13	0.26
February 25	0.10	....	....	....
March	....	0.35	0.27	0.50
March 26	0.63	....	....	....
April	....	1.03	0.81	1.00
May	....	0.61	0.54	0.83
May 24, 26-28, 30	0.77	....	....	....
May 31 to June 6	0.35	....	....	....
June	....	0.26	0.29	0.65
July	....	0.46	0.44	0.83
July 10 and 11	0.35	....	....	....
August	....	0.56	0.54	0.53
September	....	(1.24)	0.81	0.50

*The figures in parenthesis are approximations from incomplete records.

Evidently, in periods, such as October to December, 1925, and January, February, June, and July, 1926, the suspended load increases with the dis-

* *Technical Bulletin No. 8*, Univ. of Arizona, p. 7.
† *Water Supply Paper No. 596-B*, U. S. Geological Survey.

TABLE 12.—SILT DETERMINATIONS AT SAMPLING STATIONS, IMPERIAL CANAL SYSTEM, SEPTEMBER, 1917.*

Day of month.	Main Canal, near Hanlon Heading.	Main Canal, at Cudahy Check.	Main Canal, at Alamo Mocho.	Main Canal, at Lawrence Heading.	East Side Canal, at Check No. 1.	East Side Canal, at Myrtle Check.	East Side Canal, at Junction Lateral.	Central Main Canal, at 10-ft. drop.	Dahlia Canal, at heading.	Dahlia Canal, at Dahlia Canal No. 12.	Brawley Main Canal, at I. W. Co. No. 4, Del.	West Side Canal, at Wisteria Check.	West Side Canal, at International Boundary.	East Side Canal, at I. W. Co. No. 8, Del.	Trifolium Canal No. 6, at heading.	No. 5 Main Canal, at I. W. Co. No. 5, Del.	Rositas Canal, at heading, Alamo River.	North End Canal, at heading, Alamo River.
1	0.164	0.460	0.431	.....	0.426	0.450	0.456	.....	0.444	.....	0.549	0.308	0.595	0.171	.....	0.840	.....	0.453
2	0.182	0.627	0.350	.....	0.380	0.396	.....	.....	0.535	0.568	0.357	0.087	0.333	0.393	0.233	0.169	.....	0.395
3	0.164	0.350	0.336	.....	0.530	0.368	0.227	.....	0.380	.....	0.444	0.181	0.253	0.230	0.246	0.165	.....	0.856
4	0.160	0.287	0.310	.....	0.164	0.404	0.348	0.376	0.404	.....	0.404	0.079	0.119	.....	0.138	0.314	.....	0.424
5	0.211	0.236	0.270	.....	0.342	0.284	0.258	0.302	0.430	0.462	0.370	0.089	0.049	.....	.....	0.234	.....	0.798
6	0.203	0.289	0.298	.....	0.246	0.278	0.224	0.312	0.376	0.282	0.416	0.059	0.289	.....	.....	0.238	.....	0.288
7	0.516	0.357	0.213	.....	0.234	0.276	0.304	0.282	0.490	0.646	0.384	0.043	0.057	.....	.....	0.218	.....	0.800
8	0.140	0.200	0.336	.....	0.232	0.270	0.212	0.270	0.264	0.514	.....	0.306	0.027	.....	.....	0.214	0.472	0.276
9	0.192	0.225	0.262	0.196	0.216	0.232	0.184	0.250	0.418	0.396	.....	0.152	0.093	.....	0.176	0.214	0.294	0.724
10	0.188	0.225	0.225	0.242	0.260	0.266	0.230	0.274	0.442	0.406	.....	0.198	0.069	.....	0.150	0.208	0.266	0.820
11	0.074	0.416	0.250	0.194	0.234	0.360	0.240	0.240	0.372	0.324	.....	0.128	0.071	.....	.....	0.214	0.146	.....
12	0.088	0.285	.....	0.212	0.216	0.242	0.160	0.238	0.426	0.224	.....	0.010	0.053	.....	.....	0.204	0.162	.....
13	0.077	0.295	0.218	0.222	0.202	0.286	0.136	0.156	0.338	0.254	.....	0.296	0.083	.....	.....	0.200	0.240	.....
14	0.087	0.231	0.230	0.220	0.174	0.202	0.128	0.158	0.340	0.244	.....	0.174	0.139	0.112	.....	0.200	0.192	0.180
15	0.074	0.231	0.143	0.188	0.178	0.210	0.128	0.086	0.266	.....	.....	0.074	0.139	0.174	.....	0.208	0.108	0.124
16	0.077	0.151	.....	0.206	0.154	0.208	0.120	0.146	0.186	0.188	.....	0.076	0.380	0.160	0.114	0.238	0.266	0.382
17	0.144	0.149	0.141	0.150	0.138	0.108	0.092	0.282	0.208	0.260	.....	0.078	0.168	0.104	0.108	0.082	0.198	.....
18	0.127	0.310	0.111	0.226	0.168	0.120	.....	0.240	0.088	0.170	.....	0.098	0.086	0.099	.....	0.062	0.070	0.206
19	0.155	.....	0.131	.....	0.236	0.142	0.088	0.272	0.290	0.154	.....	0.082	0.130	0.170	.....	0.244	0.224	0.168
20	0.106	.....	.....	.....	0.238	0.200	0.054	0.244	0.360	0.290	.....	0.110	0.116	0.124	0.192	0.258	0.254	0.466
21	0.111	0.296	0.098	0.242	0.208	0.410	0.088	0.216	0.288	0.220	.....	0.130	0.140	0.196	0.286	0.224	0.308	0.546
22	(0.200)	0.274	0.109	0.212	0.242	0.206	0.152	0.198	0.368	0.198	.....	0.112	0.158	0.146	0.290	0.202	0.138	.....
23	(0.230)	0.316	0.054	0.232	0.278	0.198	0.142	0.198	0.296	0.180	.....	0.072	0.166	0.184	0.274	0.202	0.176	.....
24	(0.270)	0.356	0.212	0.238	0.340	0.236	0.112	0.254	0.320	.....	.....	0.116	0.132	0.150	0.286	0.256	0.260	.....
25	(0.300)	0.334	0.274	0.274	0.454	0.224	0.254	0.394	0.340	0.246	.....	0.050	0.146	0.172	0.224	0.434	0.220	.....
26	(0.600)	0.538	0.248	0.520	0.640	0.304	0.424	0.460	0.434	0.308	0.394	0.200	0.170	0.218	0.240	0.430	.....	0.418
27	0.900	0.808	0.268	0.740	0.596	0.432	0.602	0.776	0.462	0.494	.....	0.250	0.286	0.234	0.342	0.594	0.628	.....
28	(1.000)	0.904	0.494	0.802	0.948	0.808	0.754	0.782	0.922	0.658	.....	0.436	0.284	0.820	0.370	0.754	0.896	.....
29	(0.600)	1.420	0.717	0.966	0.944	0.790	(1.000)	1.116	1.118	0.722	.....	0.536	0.436	0.582	0.504	0.820	0.706	.....
30	0.259	0.939	0.876	0.854	1.006	1.202	1.122	0.772	1.226	.....	.....	0.410	0.418	0.558	0.538	0.688	1.134	.....
Average.....	[0.145] (0.240)	0.406	0.272	[0.357]	0.358	0.336	[0.262] (0.290)	0.344	0.427	0.351	[0.415]	0.165	0.186	[0.221]	[0.259]	0.295	[0.335]	[0.404]

*There is a similar sheet for each month, June, 1917, to June, 1918. Figures in parentheses were supplied by the writer. Brackets have been used to show that the reported figures need correction.

tance below the Grand Canyon; in other periods, such as March to May and August, 1926, the suspended load remains fairly constant. In September, 1926, there was even a decrease of suspended load with distance below the Grand Canyon. These results seem to confirm the conclusion that at times the river picks up material from its bed and other times it drops part of its suspended load.

In the Grand Canyon and in the various narrows below, there is coarse sand and gravel in the river bed, and the silt carried by the river is supposed to be in suspension. It was hoped, therefore, that the comparison presented in Fig. 6 might disclose some facts that could be used in estimating the river's bed load, but they give no indication, except possibly that a considerable portion thereof originates below the Grand Canyon.

The Basic Record of Silt Determinations.—The records of the determinations of silt in suspension in the Colorado River, at Yuma, are too voluminous to be incorporated in this paper. The same is true of the records of the

TABLE 13.—PERCENTAGE OF SILT IN COLORADO RIVER AT YUMA, ARIZONA, FEBRUARY, 1920.

(Mean percentage of silt, 1.35; discharge, 2 188 316 acre-ft.; silt, 21 477 acre-ft. (at 85 lb. per cu. ft.).)

Day of month.	Distance from left bank, in feet.	Depth, in feet.	Silt in Percentage by Weight.			Discharge, in second-feet.
			Surface.	Mid-depth.	Bottom.	
2	170	8.0	0.47	0.52	0.74	14 200
	350	5.0	0.45	0.61	0.64	
	500	6.0	0.55	0.80	0.69	
6	180	7.0	0.73	0.82	0.91	10 100
	360	4.5	0.68	0.78	0.82	
	510	3.0	0.73	....	0.84	
9	180	7.0	0.59	0.68	0.69	9 800
	360	4.0	0.64	0.66	0.68	
	510	4.0	0.55	0.61	0.66	
13	160	16.0	1.11	1.39	1.49	41 200
	340	12.0	1.91	1.20	1.16	
	550	11.0	1.38	1.22	1.41	
16	170	28.0	1.41	1.30	1.44	31 200
	350	9.0	1.72	1.69	1.34	
	530	7.0	1.11	1.63	1.58	
20	180	14.0	1.29	1.51	1.56	15 900
	330	9.0	1.08	1.30	1.29	
	480	4.5	1.04	1.60	1.62	
23	160	16.0	2.12	2.19	2.66	49 200
	340	12.0	2.45	2.36	1.88	
	550	8.0	2.10	2.64	2.38	
28	150	37.0	1.76	1.91	1.84	65 100
	360	26.0	2.97	1.97	1.74	
	540	12.5	2.40	3.32	2.51	

sampling results on the Imperial Canal System. Sample sheets illustrate the nature and scope of these records. These are presented as Tables 12 and 13. It is to be noted that the recomputed weighted mean for February, 1920 (Table 13), is 1.70% and the volume of silt carried is 28 000 acre-ft. at 85 lb. per cu. ft., instead of 1.35% and 21 477 acre-ft., as appear in the original record. Similar recomputations for use in this paper were made, where necessary, of all Government records at Yuma since 1919.

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PAPERS AND DISCUSSIONS

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MEASURING MATERIALS FOR CONCRETE

By William Mayo Venable,* M. Am. Soc. C. E.

Synopsis

The paper deals with those variations in workability, strength, and solidity of concrete from batch to batch which result from unavoidable variations in the water content and gradations of the aggregates as received for use. It considers especially how the different methods of measuring the ingredients tend to increase or to reduce the difficulties of securing uniform results.

The bulking of samples of sand from twenty-two localities when measured by different methods was determined experimentally in order to ascertain the best method of measuring. Briquettes were made with all these sands, using different proportions of cement and water, in order to ascertain how the method of measurement affected the uniformity of the results. Both the strength and the solidity of these briquettes were determined.

Interesting results of these tests are here discussed, including the function and practical control of the mixing water. It is shown that the chief function of the mixing water is to make it possible to pack the other ingredients of the concrete in the least possible space; and that when the quantities of cement, sand, and stone to be used in any batch have been determined, such a quantity of water should be used as will make it possible to compact the mix into the smallest volume.

Effect of Water

Mixing of concrete has been thoroughly discussed, and the basic principles are well understood, although there remain differences of opinion as to the best procedure in applying these principles in design. However, a design depends on the properties of the materials to be used; and the greatest difficulties in the design of concrete arise from uncertainties regarding these properties.

Note.—Written discussion on this paper will be closed in December, 1929.

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Concrete ready to place consists of a mechanical mixture of at least four different materials—stone or coarse aggregate, sand or fine aggregate, cement, and water. For a definite combination of perfectly dry materials and a little water only, the mixture cannot be compacted as much as when thoroughly dry. The bulk increases not because of the volume of the water, but because the water films on the grains prevent them from rolling or slipping on one another. This "bulking" at first increases with the quantity of water, until there is sufficient water to begin to lubricate the particles instead of to bind them. Thereafter an increase in water makes it possible to compact the mass more with certain mixes than can be done, even by pressure, with a perfectly dry batch. Then the voids between the grains are completely filled with water. If any more water is used, the bulk is increased accordingly.

If the quantities are measured out separately, and the fineness or the void spaces, or both, in any one of the three materials varies somewhat from batch to batch, the optimum quantity of water for maximum compactness will likewise vary. In practice it is impossible to procure stone and sand that do not vary and, consequently, there is always an element of uncertainty as to the mix. It is desirable, therefore, that the measuring methods be such as to compensate, as far as possible, for variations in the physical properties of the materials as received on the work, so that the resulting mixture has a minimum of void space to be filled by water, and has enough water—but no more than enough—to insure "workability". Such methods of measurement are available.

Most of the water used in mixing concrete is necessary only to lubricate the mass. Sometimes only 20% is required for chemical purposes, the other 80% remaining after the concrete hardens, until finally it dries out and is replaced by air. On ordinary poured concrete work, it is desirable to use only enough extra water to give maximum compaction.

Concepts of Concrete

Thus, the final mixture of stone, sand, cement, and water, may be considered in several different ways, as follows:

(*a*) A mixture of "total dry materials" with water. This does not indicate directly a desirable method of measuring each of the ingredients separately.

(*b*) A mixture of "combined aggregate" (sand and stone) with cement grout. This usually occurs first to the practical man when he attempts to interpret the water-cement ratio theory, and has not yet grasped the importance of the other materials which enter into the mix. The strength of concrete is not approximately the same as the strength of grout of the same water-cement ratio.

(*c*) A mixture of "coarse aggregate" (stone) with cement mortar. Usually experience leads to the adoption of Method (*c*).

Bulking and Segregation

Whenever granular materials are mixed with water, two phenomena may be observed, their relative importance depending on the quantity of water used, and the conditions of placing or of compacting the mix. These are (1) bulking, and (2) segregation, due to the settling or moving of large particles faster than small ones when there is too much water.

For stone the bulking due to dampness is negligible. For sand it varies with the fineness and other physical properties and with the water content, sometimes amounting to 15% for relatively coarse sand, or to 50% for a very fine sand. The moisture content for maximum bulking varies between 3% and 12% of the weight of the sand. Cement behaves like very fine sand, and bulks much more than 50% with small percentages of water; but this is negligible since cement is always measured dry, usually by weighing. Sand, however, as received on the work, almost always is moist or wet, the variation seriously affecting its practical measurement.

Segregation and its effects are best shown by simple experiments. In a glass tube, say 1½ in. in diameter and 4 ft. long, pour dry sand to a depth of about 3 in. Empty it out, fill the tube with water, and pour the sand back. The larger grains will settle faster than the smaller and the sand will reach a greater depth than in the dry.

If the same experiment is performed with cement, a similar result will be observed, except that perhaps several hours will be required for the smallest particles to settle. The cement should be mixed with some water before the final pouring, to insure that the fine and coarse grains are completely separated before settling. Allow the settled cement to harden, for a day, then remove and examine it. It will have practically no strength, the top forming merely a "mud", called "laitance", and the bottom part being soft and brittle.

Something like this occurs in mortar when an excess of water is used, but the segregation of the cement is not nearly as complete, because the distance that each particle must settle is small. A mixture of 1 volume of cement to 1.2 volumes of water makes an utterly worthless "neat" cement briquette, but a similar water-cement ratio in concrete gives considerable strength, although such a mixture is by no means recommended.

The plain fact is that to secure workability much more water is required to compact the sand and cement than to compact the cement alone. Neat cement is quite "workable" with 40% of its bulk of water, but between twice and three times this quantity is commonly required to "work" a mixture with sand and stone added.

Tamping or jarring, if it is to improve the concrete at all, does so by loosening the ingredients from positions where points rest against surfaces into positions where surfaces are in contact. It is desirable that all surplus water and air escape, leaving the grains packed together as tightly as possible, provided the water does not carry with it any cement, or cause partial segregation of the fine particles of cement from the coarse particles in the mixture. Obviously, the amount of such segregation or washing out increases with the quantity of water removed; therefore, it is desirable to use the minimum water to secure the initial condition of workability necessary for compacting the concrete. Even then the mass may not be plastic. It is only plastic when the various grains are surrounded by enough water to permit them to roll on one another without excessive surface friction.

Constant Mortar

In short, although water is necessary for workability, any excess is injurious: First, because it increases the volume; and, second, because it

causes segregation of the cement. Concrete usually, although not always, is placed under conditions which render it very difficult to remove surplus water without injurious segregation. Therefore, the practical problem is how to assure the use of only the necessary quantity of water in each batch. This is really much more important than that any one material shall be kept constant in quantity, although in every respect successive batches should be as nearly alike as possible.

In making concrete, the general practice is to measure separately four materials, water, cement, sand, and stone, and to mix them. The man doing the measuring must set each measuring device to secure the correct final result. Frequently this is difficult and it is desirable to specify the mixture in such a way that practical measurements are simple. The chief practical difficulties, with which the specifications may not have been concerned, are:

(*a*) Moisture already in the sand must either be eliminated or allowed for in measuring. This may vary excessively from batch to batch.

(*b*) The coarse aggregate may not be uniform, and, consequently, the proportion of sand to stone desired in a batch may vary. As compared with these difficulties, other problems of measurement are of minor importance.

The easiest way to overcome these difficulties practically, aside from any theoretical considerations which influence the design of the mix, is to conduct the measurement so as to produce a constant volume of uniform mortar for each batch. Then if and when necessary, the quantity of stone used therewith can be varied. Constant mortar is necessary for constant concrete, regardless of what the strength or other properties of that concrete are to be.

Sands and Their Differences

Sands commercially available for concrete work differ exceedingly in the matter of fineness and grading of particles. For the purpose of ascertaining how the physical properties affect the accuracy of various methods of measurement, the writer procured samples of commercial sands from different parts of the United States, and tested them in different ways. The sieve tests (Table 1) were conducted as follows: First, after thoroughly drying, 100 cu. cm. of each sample was measured, by pouring into a graduate, without shaking or otherwise compacting, and was weighed (Table 1, Column (3)). It was then separated by sieving, the various grades (beginning with the finest) being poured back into the graduate and the total volumes read (Columns (4) to (10)). Thus, of Sample No. 18, originally 100 cu. cm., 1.0 cu. cm. passed the finest sieve (100-mesh); 14.5 cu. cm. were retained on the next coarser sieve (48-mesh) which poured into the graduate gave a total reading of 15.5 cu. cm. The final reading was 103 cu. cm., showing that, by screening, this sand increased in volume 3 per cent.

This increase takes place with all natural sands, but differs greatly among the samples (Table 1). This is due, in large part, to differences in the shape, as well as in the size of the grains, the Pittsburgh sand with 3% increase having rounded grains, and the Boston sand with 25% increase, having grains of irregular figures with sharp edges and corners.

The differences in the relative proportions of fine grains in natural sands is shown graphically for some of the samples, in Fig. 1. Although the average diameter of grains in Sample No. 6 (Table 1) is about eight times that in

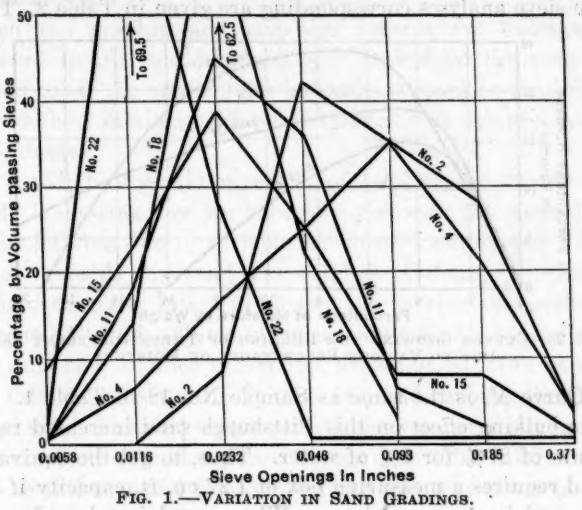

FIG. 1.—VARIATION IN SAND GRADINGS.

Sample 22, both are satisfactory for concrete. Evidently, the fineness sieve test alone, gives insufficient information to compare the suitability of various sands for concrete mortar.

TABLE 1.—SIEVE TESTS OF VARIOUS SANDS.

(Total quantities, in cubic centimeters (loose), passing various openings.)

Sample No.	Locality.	Weight of 100 cu. cm. dry, in grammes.	Sieve Openings, in Inches.						
			0.0058	0.0116	0.0232	0.046	0.093	0.185	0.371
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	St. Louis, Mo., Mississippi River	139.6	20.0	102.5	.....	.....	.....	.....	.....
2	St. Louis, Mo., Mississippi River	165.3	0	trace	8.0	50.0	85.0	106.0	109.0
3	Baltimore, Md.	158.3	0	2.0	16.0	50.0	80.0	102.0	.....
4	Ulster County, New York (bank)	162.2	1.0	8.0	25.0	50.0	85.0	113.0	116.0
5	Baltimore, Md.	166.1	0	5.0	32.0	60.0	81.0	115.0	.....
6	Boston, Mass.	180.5	1.0	10.0	33.0	62.0	83.0	102.0	125.0
7	Morrisville, Pa. (sand bank)	150.9	trace	5.0	43.0	82.0	97.0	109.0	110.0
8	Meramec River, Missouri	170.2	0.5	13.0	52.0	74.0	96.0	116.0	.....
9	St. Louis, Mo., Mississippi River	163.9	1.5	32.0	62.0	96.0	106.0	108.0	.....
10	Houston, Tex.	162.9	trace	9.0	64.0	95.0	102.0	106.0	108.0
11	Lake Wales, Fla.	158.6	0.2	20.0	65.0	101.0	107.0	109.0	.....
12	Morrisville, Pa., Delaware River	162.0	0.5	20.0	70.0	95.0	101.0	107.0	113.0
13	Birmingham, Ala.	147.7	trace	9.0	71.0	100.0	107.0	109.5	.....
14	Cow Bay, N. Y.	150.2	3.0	24.0	71.0	94.0	102.0	109.0	110.0
15	Boston, Mass.	153.4	9.0	32.0	71.0	96.0	104.0	112.0	113.0
16	Brooklyn, N. Y. (bank)	149.2	0.5	23.0	72.0	99.0	107.0	109.0	111.0
17	Baltimore, Md.	148.9	2.5	25.0	73.0	93.0	99.0	106.0	109.0
18	Pittsburgh, Pa., Allegheny River	141.6	1.0	15.5	78.0	100.0	103.0	.....	.....
19	Alafia River, Florida	151.2	1.0	15.0	80.0	100.0	103.0	105.0	.....
20	Morgantown, W. Va.	145.1	0.5	17.0	81.0	96.0	102.0	105.0	106.0
21	Baltimore, Md. (bank sand)	145.0	2.0	16.0	86.0	99.0	100.5	103.0	104.0
22	New Orleans, La., Lake Ponchartrain	146.8	2.5	72.0	100.0	100.5	100.75	102.5	103.0

The amount that sand "bulks" when moistened depends on the degree of moistness and the fineness of the sand; the finer the sand, the greater the bulking. A good idea of the extent of this bulking may be gained from Fig. 2.* The sieve analyses corresponding are given in Table 2. The Allegheny

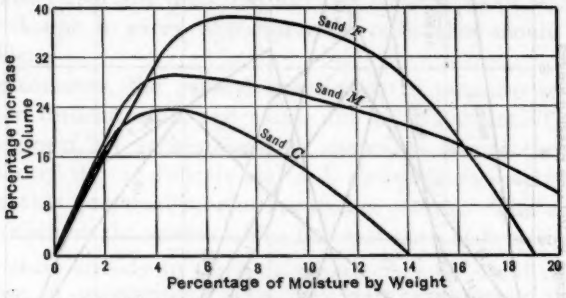

FIG. 2.—CURVES SHOWING THE BULKING OF THREE DIFFERENT SANDS DUE TO VARIOUS PERCENTAGES OF MOISTURE.

River sand, Curve *M*, is the same as Sample No. 18 in Table 1. As moisture was added the bulking effect on this Pittsburgh sand increased rapidly, reaching a maximum of 27% for 4% of water. Thus, to get the equivalent of 1 cu. ft. of dry sand requires a measuring box of 1.27 cu. ft. capacity if the sand has 4% moisture, and is measured loose. Where sand is taken from open stock piles, using volume measurement and allowing for bulking, it is generally practicable to get the specified quantity in a batch, without variation of more than 5 per cent. If, however, some of the batches are very wet or very dry, and the others moderately wet, as in a pit or bin, the variation may amount to more than 15% in the Pittsburgh District.

TABLE 2.—SIEVE ANALYSES OF SANDS BY WEIGHTS. PERCENTAGES RETAINED ON STANDARD SIEVES.

Sieve size.	Sand "C", Potomac River, concrete sand.	Sand "M", Allegheny River, concrete sand.	Sand "F", Potomac River, asphalt sand.
¼ in.	1	0	0
No. 10	48	13	1
No. 20	70	24	7
No. 30	79	48	19
No. 40	83	68	31
No. 50	88	89	49
No. 80	93	97	70
No. 100	95	98	82
No. 200	97	99	92
Passing No. 200.....	3	0.8	8

SATURATED SAND

However the measurement is made, great accuracy in each batch can be expected only if the moisture content is kept uniform, batch after batch.

* "The Bulking of Moist Sands: Effect of Phenomenon on Strength and Yield of Concrete," by Arthur A. Levison, *Public Roads*, July, 1924, Vol. 5, No. 5, p. 21.

There are two attainable conditions of uniformity. One is to use perfectly dry sand; in actual work this can be attained only by artificial drying, which is expensive. The other is to have the sand saturated with water, or to measure it under water.

Even when the grading and moisture content are constant, the total quantity of sand in the container will still depend on the tamping or compacting applied to it. It may readily be made as loose as possible, by simply pouring it into the container without shaking, or as compact as possible by jarring the container.

In Table 3 will be found the results of tests of the twenty-two samples listed in Table 1, showing how the volume varies with the method of measurement when the bulking due to variable moisture is eliminated. In each case, 100 cu. cm. of perfectly dry sand was weighed (Column (2), Table 3). The sand was poured carefully into a graduate. By jarring the graduate the sand could be compacted to the volume given in Column (3), but no more. Pouring the same sand into another 100-cu. cm. graduate, half full of water, caused it to assume the volume given in Column (4), and rapping the graduate with the submerged sand caused it to compact to the volume given in Column (5). If the sand were poured suddenly into a very deep tube, filled with water, the larger grains settled first and the final volume was that given in Column (6).

TABLE 3.—VARIATION OF SAND VOLUMES AND VOIDS BY DIFFERENT MEASUREMENTS.

Sample No.	Volume of Sand Plus Voids, in Cubic Centimeters.						Percentage of Voids.					
	Weight of 100 cu. cm. dry and loose.	Compacted by rapping.	After pouring into water.	Under water after rapping.	Maximum segregation.	Net, as determined by water displaced.	Dry.	Compacted by rapping.	After pouring into water.	Under water after rapping.	Maximum segregation.	Column (10), Column (11).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	139.6	88.0	94.0	86.0	102.0	56.0	44.0	36.4	40.4	34.9	45.1	0.864
2	165.3	98.0	108.0	98.0	116.3	70.0	30.0	28.6	35.2	28.5	39.8	0.801
3	158.3	99.0	110.0	100.0	123.0	64.0	36.0	35.3	41.8	36.0	47.5	0.861
4	162.2	99.0	114.0	102.0	123.0	66.0	34.0	33.3	42.1	35.3	46.3	0.839
5	166.1	95.0	110.0	98.0	133.7	68.0	32.0	28.4	38.2	30.6	49.3	0.801
6	180.5	97.0	112.0	100.0	124.6	70.0	30.0	27.9	37.5	30.0	43.8	0.800
7	150.9	94.0	108.0	98.0	122.8	66.0	34.0	30.0	38.8	32.6	46.2	0.842
8	170.2	96.0	107.0	94.0	120.3	66.0	34.0	31.2	38.3	30.0	45.4	0.783
9	168.9	98.0	108.0	98.0	117.6	70.0	30.0	28.6	35.2	28.6	40.8	0.802
10	162.9	94.0	110.0	98.0	120.9	68.0	32.0	27.6	38.2	27.3	45.2	0.714
11	158.6	98.0	106.0	94.0	120.5	64.0	36.0	34.7	39.6	31.3	46.7	0.790
12	162.0	96.0	108.0	98.0	119.8	66.0	34.0	31.2	38.8	32.6	44.9	0.840
13	147.7	91.0	100.0	90.0	105.8	56.0	44.0	38.0	48.8	37.7	47.1	0.732
14	150.2	91.0	110.0	98.0	127.0	62.0	38.0	31.7	43.6	36.7	51.2	0.842
15	153.4	94.0	106.0	90.0	120.5	60.0	40.0	36.1	43.4	33.3	50.2	0.767
16	149.2	94.0	104.0	94.0	114.2	60.0	40.0	36.2	42.3	36.2	47.4	0.856
17	148.9	94.0	110.0	98.0	125.0	64.0	36.0	31.9	41.8	34.7	48.8	0.830
18	141.6	90.0	100.0	90.0	107.5	58.0	42.0	35.5	42.0	35.5	46.1	0.845
19	151.2	92.0	110.0	95.0	115.8	64.0	36.0	30.4	41.8	32.6	44.7	0.780
20	145.1	95.0	102.0	92.0	117.0	60.0	40.0	36.8	41.2	34.5	48.5	0.838
21	145.0	92.0	112.0	98.0	123.9	70.0	30.0	24.0	37.5	28.5	43.4	0.733
22	146.8	95.0	108.0	90.0	112.8	58.0	42.0	38.9	46.3	35.7	48.7	0.771

Each of these five different conditions of measurement will give consistent results. That by measuring the sand loose, but completely saturated with water (Column (4)), is the simplest in practice, the measurements being reproducible on a commercial scale to within 0.5%, batch after batch, with the commercial sand used in Pittsburgh.

The net volume of sand grains only, as determined by the water displaced (Column (7)), shows that the voids vary between 30 and 44 per cent. By segregation the grains may be so arranged that more than one-half the space is void (Samples Nos. 14 and 15).

Grout to Be Added

Now suppose that just sufficient grout is to be added to fill the voids in the sand; at once the question would arise: In what way shall the voids be measured? Will the percentages in Table 3 suffice, or do these correctly represent the quantity of grout to be used? The smallest voids (when the sand is most compact) are indicated in Columns (9) and (11), and the voids of loose measurement in Columns (8) and (10). Are the voids increased beyond both these amounts when grout is added and, if so, to what extent? Without the answer to this question no determination of voids will show how much cement and water will be required to fill them properly.

Grout is not a simple liquid, but a liquid with solid grains in suspension. The amount required to "fill" the voids depends not on the voids in the sand, as measured by the water test, as much as on the volume of cement already mixed with the water to form the grout. When grout—or even dry cement—is mixed with sand, some of the particles of cement (with a diameter as large as one-eighth that of the average sand grain) prop the grains apart, thus increasing the voids to be filled, while others occupy the spaces between the grains and thus diminish the voids. Therefore a thorough mixture of perfectly dry sand and perfectly dry cement cannot be compacted so that it will have no voids. Perhaps for any commercially available sand and cement 15% of void space is the minimum.

Moist cement and sand cannot be thoroughly compacted unless there is at least enough water to fill all the voids in the compacted mass. At least 15% of the final volume, therefore, must be water, as this is necessary to overcome surface tension among the particles and thus permit them to adjust themselves to fit snugly against one another. The quantity of water required to fill the voids completely depends not on the quantity of cement, nor on the voids in the sand measured separately by any method whatever, but on the voids in the cement and sand when mixed together in that condition of closeness permissible for "workability" for placing in the forms.

Soundness of Briquettes

To ascertain the relationship between quantities of sand, cement, and water on the soundness and strength of mortar, a number of sample briquettes were made and tested, and the results are given in the accompanying diagrams.

As it was desired to investigate only the effect of varying the proportions, it was essential to use cement and sand of uniform quality. The cement for the briquettes came out of the same sack. All the washed Allegheny River sand was from the same stock pile, thoroughly dried and sifted. To secure uniformity of size only material passing a 0.0232-in. screen and retained on a 0.0116-in. screen was used.

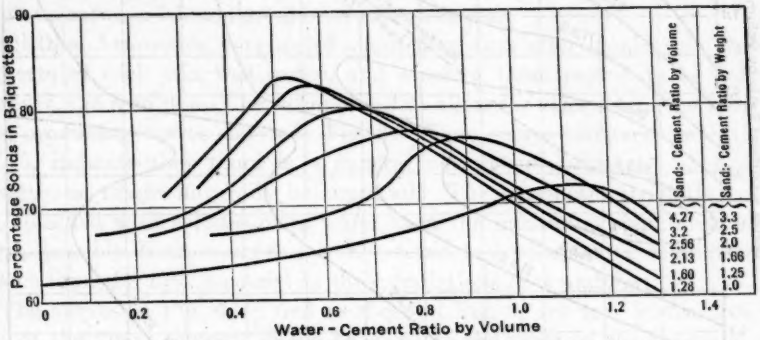

FIG. 3.—SOUNDNESS OF BRIQUETTES FOR VARIOUS RATIOS OF SAND TO CEMENT AND WATER TO CEMENT.

In turn, 100 grammes of this sand was mixed with 10 grammes, 20 grammes, etc., of cement, and with enough water to give water-cement ratios, by volume of 20%, 30%, etc. Each briquette was made in the ordinary standard mould, the materials being pressed in as tightly as possible with the fingers. It was weighed and the residual materials were weighed to check, so that the exact weights of water, cement, and sand in each briquette could be computed. It was assumed that water equal to 10% of the weight of the cement eventually combined chemically to form solid material. The percentage of solid matter to the total volume of each briquette could then be calculated.

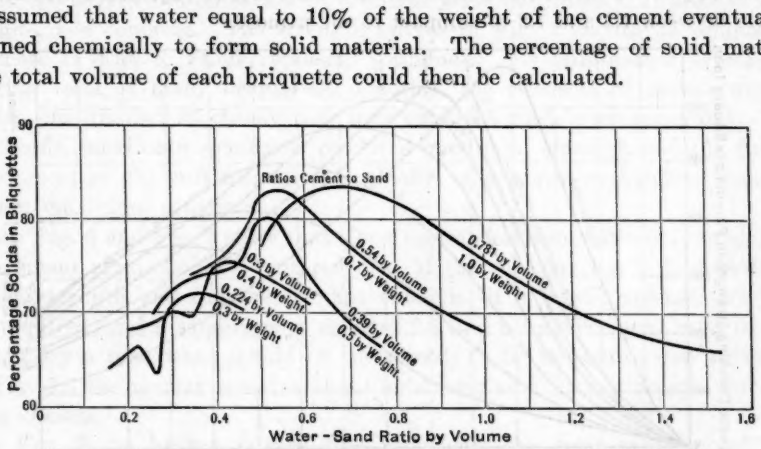

FIG. 4.—SOUNDNESS OF BRIQUETTS FOR VARIOUS RATIOS OF CEMENT TO SAND AND WATER TO SAND.

The results are given graphically in Figs. 3 and 4. With equal weights of sand and cement (Fig. 3) the soundest briquette was that containing about 52% as much cement as water, by volume. That is, it could be compacted farther in the mould than with more, or with less, water. Thus, with a sand-

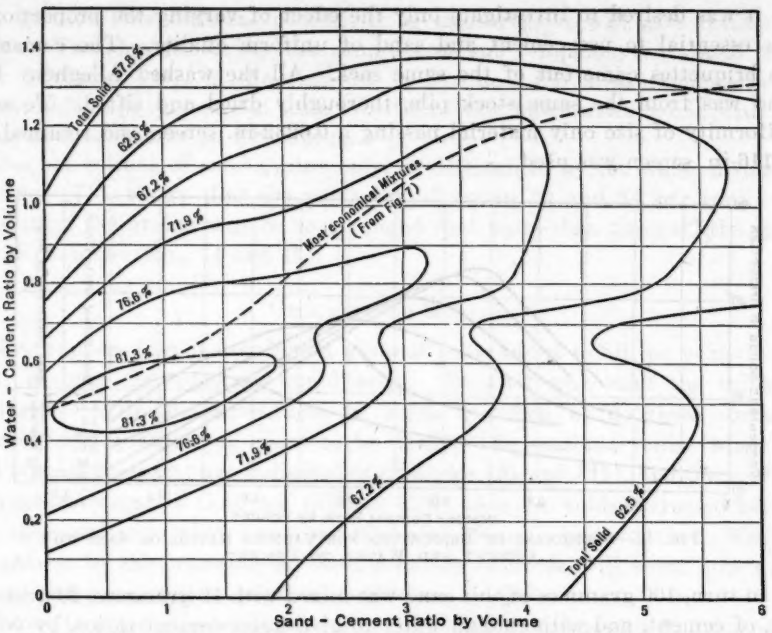

FIG. 5.—PERCENTAGE OF TOTAL SOLIDS IN BRIQUETTES OF VARIOUS MIXTURES.

cement ratio of 2 by weight, or 2.56 by volume, the water-cement ratio required to produce the densest briquette was about 0.8, while with only one-third as much cement as sand (sand-cement ratio of 4.27), it required a water-cement ratio between 1.1 and 1.2 to compact to the utmost.

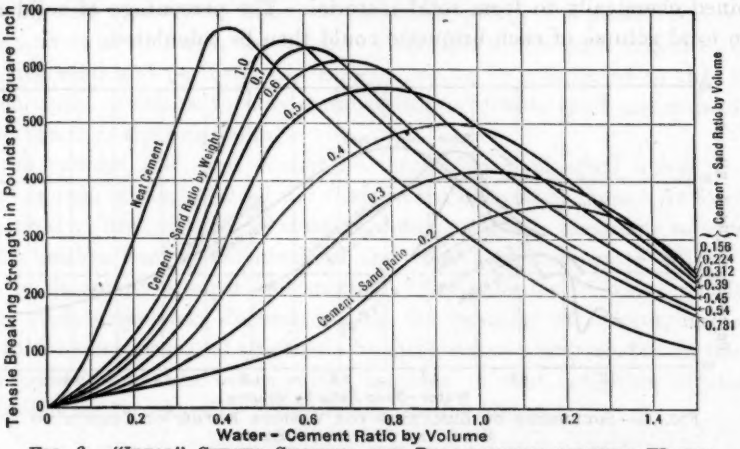

FIG. 6.—"IDEAL" CURVES SHOWING THE RELATIONSHIP BETWEEN WATER-CEMENT RATIO AND TENSILE STRENGTH FOR DIFFERENT RATIOS OF SAND TO CEMENT.

By comparison, Fig. 4 interprets the same experimental data using the water-sand ratio, instead of the water-cement ratio. Thus, a cement-sand

ratio of 0.39 takes about one-half as much water as sand to make the soundest mix, but after doubling the quantity of cement, the water must be increased from 0.5 to 0.7 of the sand to get the soundest mix.

In Fig. 5, the graphs which resemble contour lines represent mixtures of equal soundness. Thus for a mixture of 3 parts of sand to 1 part of cement, by volume, in order to make the soundest briquette, the quantity of water should be between 0.8 and 0.9 the volume of cement.

All these briquettes were tested about fifty days after moulding. Only one briquette of each mix was tested, and some of these proved to be defective. However, the results may be rationalized to allow for vagaries in the individual tests, producing curves shown in Fig. 6. These curves compared with those in Fig. 3, indicate that there is a general parallelism between soundness and strength, as might naturally be supposed. The maximum strength, however, was obtained with a little more water than the maximum soundness, as computed, possibly because more water entered into chemical combination with cement than the 10% assumed in the calculations of soundness.

The curves in Fig. 6, as well as those in Fig. 3, are very instructive. The part of the "neat cement" curve (Fig. 6) to the right of the ordinate, 0.6, is hypothetical, as corresponding mixtures cannot be made into briquettes without the cement settling and some of the water rising to the surface. It is clear, however, that with any specified water-cement ratio there is an optimum quantity of cement and sand that will give the greatest strength.

Strength Tests

Another aspect of the same relationship is shown in Fig. 7. Discrepancies between the curves and observations are mostly for the mixes that are too wet for high-grade concrete. The strength curves in Fig. 7 should be compared with those in Fig. 5, which represent soundness. By eliminating errors by averaging tests of many briquettes, doubtless the two sets of curves would resemble one another so closely that they could be used interchangeably. In other words, maximum soundness produces maximum strength and the function of most of the mixing water is chiefly to produce workability enough to attain maximum soundness.

Both Fig. 5 and Fig. 7 show that when the sand-cement ratio is altered, the water-cement ratio should be altered also, if the sole purpose is to maintain uniform strength and soundness. For example, if a tensile strength of 460 lb. is required, and a sand-cement ratio of 2.5 and a water-cement ratio of 0.8 give too dry a mix, what should be the remedy? By increasing the water to 0.9 (Fig. 7), the mortar remains about as strong or a little stronger without adding cement.

In Fig. 7 the broken curved line represents the crest or ridge of the contours. Mixtures above this line are so wet that they may be poured about reinforcing steel with little or no tamping; those below require tamping or pressure to secure the strength recorded in the tests. This line is reproduced in Fig. 5, for comparison. Any mixture below this line will have its strength increased by adding water only; any mixture above it will increase in strength by reducing the water content, providing in each case the tamping is sufficient.

Mixtures of a specified strength will be found only within the corresponding contours. Obviously, the cheapest mix is the one requiring the least cement. Therefore, the proportions of the cheapest workable mix are found at the intersection of the broken line with the contour line representing the strength required.

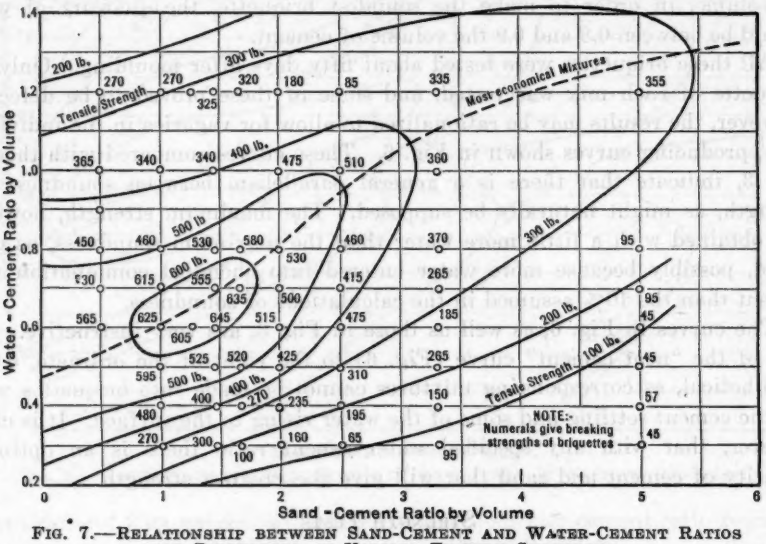
Water - Cement Ratio by Volume
Sand - Cement Ratio by Volume
Most economical Mixtures
Tensile Strength
NOTE:-
Numerals give breaking strengths of briquettes

FIG. 7.—RELATIONSHIP BETWEEN SAND-CEMENT AND WATER-CEMENT RATIOS FOR BRIQUETTES OF VARIOUS TENSILE STRENGTHS.

The use of the water-cement ratio for determining the quantity of water is, of course, arbitrary. A water-sand ratio plotted from the same data (Fig. 8) is just as applicable. If the sand were supplied in perfectly dry condition, so

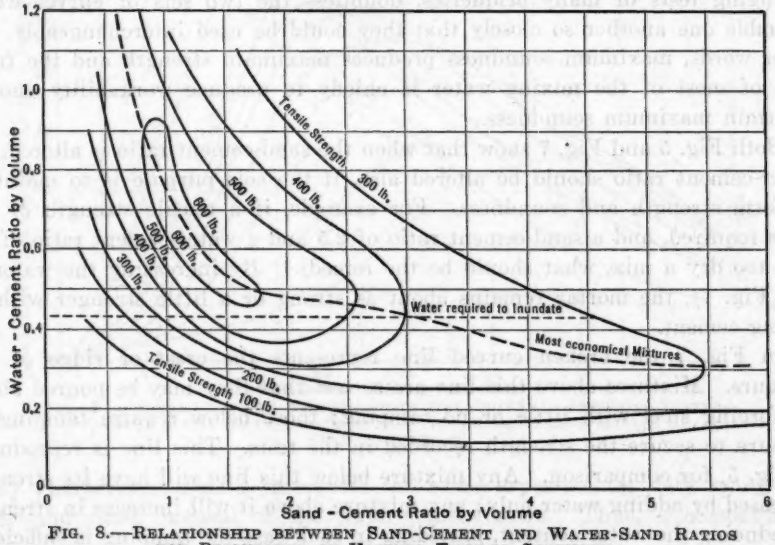
Water - Cement Ratio by Volume
Sand - Cement Ratio by Volume
Tensile Strength
Water required to Inundate
Most economical Mixtures

FIG. 8.—RELATIONSHIP BETWEEN SAND-CEMENT AND WATER-SAND RATIOS FOR BRIQUETTES OF VARIOUS TENSILE STRENGTHS.

that all the water may be measured separately, Figs. 7 and 8 are equally convenient. If, however, the sand is not dry, and not uniform, some advantages, as will be shown, accrue from the use of the form of Fig. 8.

All the tests on which Figs. 2 to 8 are based were made with a specially prepared sand, to secure such uniformity of materials that the effects of varying proportions could be observed, for theoretical purposes.

Effects of Constant Water-Cement Ratio

Obviously, the sand that, measured by bulk, requires the least cement to make a mortar of given strength and soundness is (at equal unit prices) the most economical, as cement is ordinarily the most expensive ingredient in concrete. To check the relative values of the twenty-two sand samples of Tables 1 and 3, one briquette was made with each sample. Enough water was taken to fill the voids in the sand measured inundated (Table 3, Column (10)), plus 5%, to insure perfect workability; and enough cement was used to give a water-cement ratio of 0.8 in each briquette. The sand was measured dry; the required water was calculated from the tests given in Table 3; and the volume of cement was calculated from the water, and measured by weight to avoid errors due to bulking in very small samples.

TABLE 4.—Strengths of Briquettes Made from Various Sands Using a Constant Water-Cement Ratio of 0.8.

Sample No. (1)	Locality. (2)	Percentage of voids, in inundated sand. (3)	Weight of 100 cu. cm. sand, in grammes. (4)	Weight of mixing water, in grammes. (5)	Weight of cement, in grammes. (6)	Percentage of bulk cement, in finished briquette. (7)	Weight of briquette, dry, in grammes. (8)	Breaking strength, in pounds per square inch. (9)	Cement-sand ratio by volume. (10)
1	St. Louis, Mo., Mississippi River	40.4	139.6	39.9	92.8	36.7	117.5	470	0.499
2	St. Louis, Mo., Mississippi River	35.2	165.3	39.9	92.8	33.3	128.5	560	0.499
3	Baltimore, Md.	41.8	158.3	48.3	112.3	37.6	127.8	520	0.604
4	Ulster County, New York (bank)	42.1	162.2	50.4	117.2	37.7	128.0	655	0.630
6	Boston, Mass.	37.5	180.5	44.1	102.5	34.8	130.0	565	0.551
7	Morrisville, Pa. (sand bank)	38.8	150.9	44.1	102.5	35.7	128.3	555	0.551
8	Meramec River, Missouri	38.3	170.2	43.0	100.0	36.3	129.1	540	0.538
9	St. Louis, Mo., Mississippi River	35.2	163.9	39.9	92.8	34.8	129.1	615	0.499
10	Houston, Tex.	38.2	162.9	44.1	102.5	36.2	126.3	495	0.551
11	Lake Wales, Fla.	39.6	158.6	44.1	102.5	37.2	126.3	592	0.551
12	Morrisville, Pa., Delaware River	38.8	162.0	44.1	102.5	36.7	127.1	525	0.551
13	Birmingham, Ala.	48.8	147.7	46.2	107.5	40.1	125.2	545	0.578
14	Cow Bay, N. Y.	43.6	150.2	50.4	117.2	39.9	123.9	515	0.630
15	Boston, Mass.	43.4	153.4	48.3	112.3	40.0	124.6	550	0.604
16	Brooklyn, N. Y. (bank)	42.3	149.2	46.2	107.5	39.1	123.3	435	0.578
17	Baltimore, Md.	41.8	148.9	48.3	112.3	38.7	124.1	585	0.604
18	Pittsburgh, Pa., Allegheny River	42.0	141.6	44.1	102.5	38.8	124.2	600	0.551
19	Alafia River, Florida	41.8	151.2	48.3	112.3	38.7	126.3	530	0.604
20	Morgantown, W. Va.	41.2	145.1	44.1	102.5	38.3	123.2	545	0.551
21	Baltimore, Md. (bank sand)	37.5	145.0	44.1	102.5	35.7	124.4	590	0.551
22	New Orleans, La., Lake Ponchartrain	46.3	146.8	52.5	122 0	41.6	123.7	590	0.656

In Table 4 will be found the data for each briquette and its breaking strength after the same period of curing. As all these mortars were thoroughly "workable", they should show approximately equal strength, if maintaining a

constant water-cement ratio could produce that result, regardless of the differences in the sand. From the percentages of cement to sand, by volume (Table 4, Column (10)) it will be seen that almost all these mixtures were richer than 2 of sand to 1 of cement.

TABLE 5.—STRENGTHS OF BRIQUETTES MADE FROM VARIOUS SANDS USING A CONSTANT CEMENT-SAND RATIO OF 0.4.

Sample No. (1)	Percentage of water, by volume, in sand. (2)	Water-cement ratio, by volume. (3)	Breaking strength of briquette, in pounds per square inch. (4)	Weight of dry briquette, in grammes, before breaking. (5)	Sample No. (1)	Percentage of water, by volume, in sand. (2)	Water-cement ratio, by volume. (3)	Breaking strength of briquette, in pounds per square inch. (4)	Weight of dry briquette, in grammes, before breaking. (5)
2	35.2	.88	400	129.7	12	38.8	0.98	370	124.0
4	42.1	1.05	475	126.8	13	48.8	1.22	400	118.4
6	37.5	0.94	440	125.9	14	43.6	1.09	475	124.0
7	38.8	0.96	425	122.5	15	43.4	1.08	395	121.9
8	38.3	0.94	430	127.0	16	42.3	1.06	330	119.6
9	35.2	0.88	440	127.7	17	41.8	1.05	440	122.5
10	38.2	0.95	545	124.8	20	41.2	1.03	460	118.2
11	39.6	0.99	365	122.4	21	37.5	.904	360	116.7
					22	46.3	1.16	400	116.1

CONSTANT CEMENT-SAND RATIO AND ITS RESULTS

In Table 5 are given the strengths of a series of briquettes made of a mixture of 2½ parts of submerged sand without any extra water, and 1 part of cement (both by volume), or a cement-sand ratio of 0.4; in other words, the table indicates the result of ignoring the water-cement ratio altogether, using in every case not a constant quantity of water, but the quantity necessary to saturate the sand. The water-sand ratio in Table 5, Column (2) is taken from Table 3, Column (10), and the water-cement ratio in Column (3) is computed therefrom.

In general, all these mixes are leaner in cement, than those in Table 3; they have from 80 to 60% as much cement, and less actual water, although the water-cement ratio is higher. Of course, since only one briquette was made for each test, no individual result can be considered reliable. It is remarkable, however, that the variation in breaking strengths of briquettes made by using constant proportions of inundated sand and cement, by volume, as indicated in Table 5, are so nearly alike.

NATURAL VARIATIONS IN SANDS

When sand has been measured under water, extra water may be added in any desired quantities. Table 6 gives the results of tests of briquettes all made with the same carefully screened sand, measured under water, and mixed with various quantities of cement, with extra water added in definite quantities. It shows clearly how the strength falls off usually as more water is added, when

the mix is lean (Column (3)), but that when the mix is very rich (Column (6)), water must be added to make it workable enough to obtain sound briquettes.

A similar effect occurs with natural sands used in concrete, the chief difference being that there it is necessary to begin adding water with leaner mixes. These tests were made to throw light upon the effect of moderate variations in the quality and the dryness of materials in order to ascertain the best methods of measurement in the actual work—not in the laboratory only.

Now, if the sand varies in physical properties and hence in total void spaces per measured batch, an element of uncertainty is introduced. The method of measurement may be such as to magnify or minimize the importance of this; and, obviously, the latter will be the best to adopt.

Using six successive batches of Pittsburgh sand, which were dumped out and weighed after measurement, the maximum variation in the weight of the batch was 0.5%; and the maximum variation in the water content was 2 per cent. The mean variation was much less, two batches (1 800 lb.) weighing within 1 lb. of each other. Thus, in the six batches the voids varied not more than 2 per cent.

TABLE 6.—STRENGTH, IN POUNDS PER SQUARE INCH, OF BRIQUETTES MADE WITH INUNDATED SAND.

Percentage of excess water.	Strength, in Pounds per Square Inch, of Briquettes Made with Inundated Sand. Percentage of Cement by Bulk.					Percentage of excess water.	Strength, in Pounds per Square Inch, of Briquettes Made with Inundated Sand. Percentage of Cement by Bulk.				
	33%	40%	50%	60%	100%		33%	40%	50%	60%	100%
(1)	(2)	(3)	(4)	(5)	(6)	(1)	(2)	(3)	(4)	(5)	(6)
0	550	560	530	635	*	30	.....	.....	.....	450	640
6	350	445	525	560	*	36	.....	.....	.....	.....	530
12	345	465	480	.....	*	42	.....	.....	.....	.....	550
18	.....	300	445	.....	595	48	.....	.....	.....	.....	420
24	.....	.....	325	465	600	54	.....	.....	.....	.....	475

* Too dry to be workable.

WATER REGULATION

If, however, the sand came from different sources, the variation would be greater. Then, it would be best to ascertain the extra water to be added for each sand. However, for such differences as may occur from batch to batch it is important to realize that varying the water in saturated sand in proportion to the voids is beneficial. The variation in the sands in Table 5 was extreme, but the results, except for those samples which contained deleterious materials, were remarkably uniform. Measuring the sand under water tends to produce more uniform results in the mortar than weighing the batches dry and adding uniform quantities of water.

Having determined by computation and experiment the proportions of materials that produce concrete of any desired strength and consistency, and

in order to use the sand available (which may vary in moisture content, fineness, and voids), for producing concrete that is uniform, batch after batch, it is essential to:

(*a*) Eliminate the effect of moisture already in the sand on the accuracy of measurement; and,
(*b*) Regulate the quantity of water used in each batch so as to maintain the workability of the mix with as little excess water as practicable.

Practical Measurement of Sand

Sand may be measured by bulk or by weight; moist as received on the job; dry or saturated. Some years ago the usual method was by loose bulk as received; that is, moist. There was a time when ingredients for test specimens upon which specifications were based, were measured loose and moist. For some years, however, laboratory tests have been made generally with dry rodded sand, corresponding in compactness to Column (3), Table 3, and often from 15 to 30% more compact than the same sand as received on the job. Thus, the old specification 1 : 3 : 5 with loose sand would correspond to 1 : 2.5 : 5 with dry rodded sand (laboratory conditions). Much confusion resulted and not a little injustice was done contractors by adhering to the specifications for loose measurement in the field while demanding results and costs based on rodded measurement in the laboratory.

Happily, this discrepancy has been recognized and for the most part specifications are now drawn to permit measurements on the work corresponding to those used in the laboratory. If bulk measurement is permitted, allowance for bulking of sand in the measuring containers is usually permitted also. When the bulking is determined experimentally, the error introduced is rarely more than 5% in the total quantity of sand.

Such a variation in a batch is not in itself a serious matter ordinarily, if the other materials remain constant. Thus, from Fig. 7, with the water-cement ratio constant a variation of 5% in the sand-cement ratio will have scarcely any effect on the ultimate strength. Again (Fig. 8) with the water-sand ratio constant a variation of 5% in the sand-cement ratio will have small effect on the strength. There is a more serious error, however, if the water contained in the sand varies considerably. It may be 3 to 8% of the total weight, or 7 to 20% (a difference of 13%) of the total volume of the sand, while the bulking may remain almost constant. Therefore, the water-cement ratio may easily vary by 30% when the sand-cement ratio varies only 5 per cent. Thus, the variation of 30% in the quantity of water that may enter a batch is excessive; some batches will be too wet, and others too dry. The concrete cannot be handled uniformly; and, moreover, one batch may be twice as strong as another.

The facts that only in occasional batches will the wide variations occur, and that by care in handling (avoidance of pits where sand may become water-logged) a fair degree of uniformity may be attained, render it possible to use moist bulk measurement for sand with fairly good results. On the larger and better conducted jobs the water content is checked frequently, and

allowance is made in measuring additional water to be used. This, however, is subject to serious error from batch to batch.

Weighing is slightly more accurate than volume measurement, with allowance for bulking. An extreme variation of from 3 to 8% in moisture content, by weight, means only 5% in the sand; and making allowance for moisture by tests from time to time reduces the inaccuracy. Weighing makes it impossible to have those extreme variations in actual quantity of sand—15 to 30%—that may occur in bulk measurement when some of the sand is moist, but some is water-soaked, or practically dry. However, it cannot compensate for using a variable quantity of water due to ordinary fluctuations in the water content of moist sand, which are always appreciable and sometimes excessive with material exposed to the weather.

Saturating the Sand

Accurate measurement requires that sand shall be either dried out, or saturated with water at the time of measurement. This is essential, not only for accurate measurement of the sand itself, but more particularly, by ascertaining how much water goes into the measured batch with the sand for determining the proper quantity of water to be added.

Drying sand is both troublesome and expensive, and measuring it moist introduces a probable error of 15%, or more; whereas measuring it under water gives a constant volume of sand and of water per batch. Hence, this latter method is by far the best. The additional water required must be determined, and then kept constant.

In the case of the Pittsburgh sand (Table 3, Sample No. 18), the sand measured loose under water (Column (4)) has the same bulk as if measured dry. This is true for Sample No. 13 also. Other samples, however, differ in this regard, so that it becomes necessary to determine the proportions on the basis of volume in the state actually adopted in the measuring devices, and not on any other.

The percentages of voids when sand is submerged (Table 3, Column (10)), may readily be determined. However, although this percentage varies from 35.2 to 48.8, of the twenty-two samples only two have voids exceeding 43.6 per cent. Therefore, usually the quantity of water required to saturate will be less than that in Fig. 8; and ordinarily it is necessary to add water in excess of that included with the saturated sand, in order to keep the mix workable when cement and stone are introduced. The containers used in the tests in Table 3 were small; larger apparatus would give percentages of voids somewhat less.

Batches to Capacity of Mixer

Where concrete is placed in forms, the volume turned out by the mixer frequently exceeds the amount for which forms can be prepared, and the mixer need not be run to capacity, or need be operated only part of the time. For other structures, such as massive masonry construction or concrete roads, the job capacity frequently is fixed by the mixer capacity. In any event, however, it is most economical to run the mixer at its full capacity when

running at all. The best procedure in measuring materials to mix full-sized batches is as follows:

Determine from the specifications, as nearly as possible, how much sand per batch may be used to work the mixer to capacity. The measuring device should then be adjusted to fit, and such variations as are necessary to get just the mix desired should be made by adjusting the stone (measured by bulk, with or without compensation for variable voids), the cement (measured by weight), and the excess water (measured by bulk). These adjustments are much easier to make than changes in the sand-measuring device (which must be water-tight) and permit the mixer to run at nearly full capacity.

If, however, sack measurement for cement is required, the sand container must be set for the quantity corresponding with the number of sacks of cement in the batch, after determining what volume of sand in the saturated condition is compatible with the proportions specified.

Equivalence of Dry and Saturated Sands

The actual procedure must depend on how the specifications are worded, or interpreted. The method will be illustrated by assuming several different kinds of specification:

(*a*) Suppose the specifications, based on the use of saturated sand, read:

"The sand shall be measured by bulk saturated with water, and the water filling the voids shall be included in the batch. There shall be used in each batch * * * pounds of cement for each cubic foot of sand as measured. Surplus water, constant in quantity for each batch, shall be added, as may be determined by the engineer to be necessary to secure the desired workability.

"To the sand, cement, and water specified in this section shall be added the coarser aggregate or aggregates (there may be several) elsewhere described."

A specification thus drawn leaves the workability entirely in the control of the engineer. It is entirely fair and can be enforced without hardship. There will be no "over-run" or "under-run" of cement unless there is dishonesty, or the coarse aggregate departs from specifications as to voids.

(*b*) If the specification is based on laboratory tests with sand measured in a "dry rodded" condition, it may read:

"The proportions specified for sand, cement, and water entering the various grades of concrete are based upon dry rodded sand of the grade specified (which should be the same as the sand commercially available for the work). In measuring the sand, correction must be made to insure that the actual quantity, regardless of the moisture in it, shall be equivalent in weight to the quantity specified, were it measured in a dry rodded condition."

Then it is easy for the engineer, or the contractor, to calculate the number of cubic feet of saturated sand that are exactly equivalent to dry rodded sand. The procedure is as follows:

Pack into a water pail (of Weight *a*) as much perfectly dry sand as can possibly be gotten in, by tamping or rodding. The weight of the bucket and sand, even full, will be Weight *b*. Empty out the sand so that none will be lost, pour it back slowly after the bucket has been a little more than half filled with water, allowing the surplus water to overflow. Be careful not to shake the bucket nor to waste any sand. When the bucket is exactly level

full of sand and water, the weight of the remaining sand will be c and that of the pail and saturated sand will be d. Weigh the bucket full of water only (Weight e). Then,

$$\text{Weight of bucketful of rodded sand} = b - a;$$

$$\text{Weight of sand only in bucket of sand and water} = b - a - c;$$

$$\frac{\text{Volume of saturated sand to be taken}}{\text{Volume of rodded sand specified}} = \frac{b - a}{b - a - c}$$

The weight of water included in the saturated sand is $d + c - b$, and the proportion of voids in the sand (filled with water) is $\frac{d + c - b}{e - a}$.

If the process of measuring saturated sand is carried out properly the ratio of water to saturated sand is practically constant, whether measured in a 4-qt. pail, or in a receptacle 1 cu. yd. in capacity.

(c) The specifications are based on measurements of perfectly dry sand, shoveled loosely into a measuring box.

In this case, proceed exactly as in Case (b), using loose sand instead of rodded sand for the first two weighings.

(d) If the specifications state "The sand shall be measured by bulk, in the condition received on the work, loosely shoveled into a measuring box of known cubical contents", they are inaccurate. It would pay the contractor to have sand either dried or water-soaked before delivery, if such specifications are to be enforced.

(e) The specifications call for the sand to be weighed with or without an allowance for moisture content. Suppose that "The mixture shall consist of sand, cement, water, and stone, the sand and cement to be mixed in the proportions of 2 to 1, by weight". In this case, if saturated measurement is permitted (as it ought to be), proceed as follows:

Take a bucket, Weight a, and a quantity of perfectly dry sand, Weight b, more than sufficient to fill it. Half fill the bucket with water, and then fill it with sand until it overflows. Weigh the dry sand left over, Weight c. Finally, empty the bucket and fill it with water, calling the weight, d. Then,

$$\frac{d - a}{62.4} = \text{volume of bucket in cubic feet}$$

$$\frac{62.4\ (b - c)}{(d - a)} = \text{dry weight of 1 cu. ft. of saturated sand}$$

Let r = the specified ratio of sand to cement, by weight (in the example stated, $r = 2$). Then, for every cubic foot of saturated sand used, take $\frac{62.4\ (b - c)}{(d - a)\ r}$ lb. of cement.

If cement is to be measured by the sack (of 94 lb. net), for every sack of cement take $\frac{r \times 94 \times (d - a)}{62.4\ (b - c)}$ cu. ft. of saturated sand.

The ratio, $\frac{62.4\ (b - c)}{(d - a)}$, or its reciprocal, can be determined once for all, and it will not vary as much as the probable fluctuation in the water content of the sand as delivered on the job. Consequently, this method is more accurate than weighing the sand as the supposed specifications require. It is, in fact, a method of weighing which avoids the use of scales by employing the principle of specific gravity and approximate constancy of voids when sand is submerged.

Measured by Weight

Practical constructors who have used bulk cement by weighing, report that physical properties of brands, all passing standard specifications, cause a very considerable difference in the readiness with which various cements flow through gates when being measured. The finer ground material arches more quickly under pressure and is more dusty when the arches break. When mixed with air only, the dry bulk of a given weight varies exceedingly, not only with a different, but with the same, brand of cement.

Cement differs so much in volume, according to the degree of pressure applied and time allowed in compacting, that this type of measurement should never be used. Cement should always be measured by weight. As a sack of cement is filled at the factory by automatic weighing machinery, the contents may be taken as 94 lb., and as equal to 1 cu. ft. With the sack as a unit, specifications of 1 : 2 : 4 mean 1 sack of cement, 2 cu. ft. of sand, and 4 cu. ft. of stone, or multiples of these quantities, to each batch.

With cement shipped in bulk, it is necessary to weigh out each batch independent of the cubic foot as a unit. Weighing also enables the batch to be made of a size to work the mixer to capacity. This distinct advantage recommends the weighing of cement into batches even when it is received in sacks.

The Measurement of Stone

Because the sand in the mortar props the stones apart, the voids as determined by water test will always be smaller than the spaces actually to be filled with mortar to produce sound concrete. Experiment will show, however, how much the necessary increase should be. The amount of mortar required depends in part on the fineness of the sand, and in part on the fineness of the stone. In any event, one thing is certain—the greater the percentage of voids in the stone the more mortar will be required to fill them; and the most economical sample of stone or gravel will have the smallest percentage of voids, provided the smallest fragments of the stone are not too small.

The stone may be measured either by bulk or by weight, for it does not vary in volume when moist, as does sand. Stone may vary considerably, however, in the shape and size of fragments, and consequently in the voids. The percentage of voids in coarse aggregate by water test may vary from a maximum of 55 to a minimum of 35 per cent. Assume, merely for illustration of the relative merits of batching by bulk and by weight, that stone received on a given job varies from 40 to 50% in voids, by water test, and weighs in the solid, 200 lb. per cu. ft.

The proportioning must be based on filling all the voids in the coarsest stone—50% of the stone volume, plus whatever increase is due to the size of the sand grains. Table 7 shows the actual voids, when the percentage of voids varies.

TABLE 7.—VOLUMES OF BATCH, SOLID STONE, AND VOIDS, IN CUBIC FEET.

Batch.	Percentage of Voids in Stone.		
	50	45	40
(a) Batch of 2 000 Lb.:			
Actual volume of batch	20	18.8	16.6
Solid stone	10	10	10
Voids	10	8.8	6.6
(b) Batch of 20 Cu. Ft.:			
Actual volume	20	20	20
Solid stone	10	11	12
Voids	10	9	8

Of course, there must also be some addition for the propping action of the sand grains, to be determined by experiment, but it is evident that when bulk measurement is used, and the stone has 40% voids, the batch has 2 cu. ft. more solid stone, and still is more workable than one with 50% voids. Clearly, therefore, where fixed proportions are specified, it is better to measure the stone by bulk rather than by weight, and thus to place a premium on securing the most suitable material. To measure stone by weight when it is paid for by the ton tends to produce concrete of equal cost per yard, but not necessarily of equal quality, and certainly not at the lowest expenditure in finished product for value received.

PROPORTIONING MORTAR FOR VARYING STONE

There are several ways of varying the quantity of stone from batch to batch so as to have the batches of stone equal not in volume nor in weight, but in voids.

One is to require the stone to be screened into several grades, to measure fixed quantities of each grade separately, and finally to mix these grades. The providing of several different grades of stone at the job, and a corresponding number of separate volumetric batches has much to recommend it. In many cases, however, it is more economical in the long run to use this aggregate as it comes from the crusher or the washing plant, and to include more mortar per batch rather than to go to the expense of special preparation of stone. When it has been stored in large stock piles, there may be considerable variation in voids owing to segregation caused by the larger fragments rolling down and collecting at the bottom.

When it is not desired to separate the stone and measure each grade separately, it is practicable to allow for variations in voids by combining bulk measurement with weight measurement. The principle is this: A certain amount of mortar is sufficient for a batch of stone having maximum void

spaces possible from the source of supply. Suppose, that this maximum is 50%, and that it is permissible to use such stone in a 1:2:4 mix on a given job. However, when the stone is better graded, it would be allowable to use more of it, say 1:2:4.5, because the voids are less. The problem then is, how to distinguish between the proper mixes from batch to batch as the work is carried on.

This is readily done by using a bulk measuring batcher mounted on scales. It is set for 4 parts of stone, by volume. When filled, the stone is weighed, the scales indicating automatically how much additional stone may then be introduced, by weight. On a large job quite a saving can be effected in this way. The owner's inspection is concentrated at the batching and mixing plant, instead of at the quarries, screens, and bins. Further, it is to the contractor's interest to procure the best graded stone possible because the better it is, the larger quantity he can use, with consequent saving of the more expensive material, cement.

The Size of the Job

In considering the best method of measurement for any particular piece of work, the size of the job is important. It is not always economical to go to the expense of providing accurate measuring devices, for on a small job it may cost less to allow for variability in the materials by using more cement, and by taking greater care, especially in the quantity of water used, even with crude devices. The larger the work, however, the greater will be the saving in labor, time, and the total cost of materials secured by using the most efficient and accurate devices, while the cost of installing the necessary equipment becomes proportionately less. Every project presents its own problems, the use of equipment already on hand or available being one of the factors to be considered. It is believed, however, that the point of view here presented for the very practical problem of measuring materials for batch mixtures will be helpful even in many cases where, for good reasons, it is not deemed wise to resort to the most accurate methods.

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PAPERS AND DISCUSSIONS

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ANALYTICAL SOLUTION OF MASONRY DOMES: UNBALANCED LOADS

By David C. Coyle,* M. Am. Soc. C. E.

Synopsis

This paper is a continuation of the theory of masonry domes, the elementary formulas of which are given in a paper† previously published by the Society. The following discussion deals with the more advanced theory involved in the problems of partial loadings and wind loads.

The assumptions involved are necessarily in the nature of very rough approximations, but in the investigation of unbalanced loads the aim is not so much to get an exact knowledge of the actual stresses as to obtain a theoretical assurance that in the case of some particular dome these stresses may safely be neglected. The formulas based on roughly approximate assumptions will give results which indicate the order of magnitude of the stresses in question and, in this way, serve as a guide to the judgment in unusual or extreme cases in which previous experience cannot be called upon for guidance.

Following a summary of the notation and the elementary formulas from the previous paper, the discussion deals with four cases: (1) Live load on one-half the dome; (2) live load on the center part only; (3) live load on the haunch only; and (4) wind load.

Notation‡

The following is the general notation used:

A = meridional stress, in pounds per foot of horizontal section, for dead or balanced loads.

Note.—Written discussion on this paper will be closed in December, 1929.

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† "Analytical Solution of Masonry Domes," by David C. Coyle, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 102.

‡ *Loc. cit.*, p. 103.

A_L, A_W, A_T = same meridional stress for unbalanced live load, for wind, and for total loads.

B = ring stress, in pounds per foot of profile.

B_L, B_W, and B_T are analogous to A_L, etc.

C = tension in the top ring of a vertical support, or compression in the bottom ring of a vertical lantern, in pounds.

C_L, C_W, and C_T are analogous to A_L, etc.

V_L, V_W, and V_T = shear on the horizontal or vertical planes at any point, due to unbalanced live load, wind, and total, respectively.

W = sum of all balanced loads above a given level, x.

p = balanced load per square foot, at level, x.

G = sum of all partial live loading above level, x.

g = partial live load per square foot, at x.

F = sum of all wind pressures above level, x.

f = wind load per square foot, at x.

x = distance of any point above reference plane.

y = abscissa from axis of dome (plus to the right).

z = horizontal ordinate perpendicular to y (see Fig. 1).

R = radius of curvature of profile at x.

r = horizontal distance from x to axis of dome.

θ = angle between radius, R, and horizontal.

H = radial bursting force on a circle of radius, r, for an area of 1 ft. of circumference times 1 ft. of profile, in pounds per square foot.

e = eccentricity of an unbalanced load.

The following notation applies to pyramoidal domes:

R = radius of curvature of the profile on a diagonal, at x.

D = semi-diagonal of a horizontal section at x.

d = semi-diameter of a horizontal section at x.

n = number of sides of polygonal horizontal section.

L = side of a polygon on a horizontal section at x.

ϕ = angle between diagonal and side of polygon.

M = bending in a horizontal element at level, x, in foot-pounds per foot of profile.

The following notation applies to spherical domes:

S_0 = height from top of dome to level, x.

p = uniform weight per square foot of shell.

g = uniform live load per square foot on loaded area.

f = uniform wind per square foot on the entire surface of a dome.

h = height from center of sphere to point, x ($h = x$ in Fig. 1).

V = weight of lantern, in pounds.

w = weight of shell per cubic foot.

t = thickness of shell, $= \frac{p}{w}$.

The following notation applies to conical domes:

a = vertical height of apex of cone above x.

h = vertical height of top of masonry above x.

s = slant height of top of masonry above x.

r_0 = horizontal radius at top of masonry.

p = uniform weight of shell per square foot.

g and f = uniform live loads and wind per square foot.

Elementary Formulas for Balanced Loads*

Conoids.—The ring tension at the level, x, equals

$$B = \frac{W}{2\pi R}\sec^2\theta - p\,r\tan\theta \quad \ldots\ldots (1)\dagger$$

The meridional compression at the level, x, in pounds per horizontal foot of circumference, equals:

$$A = \frac{-W}{2\pi r}\sec\theta \quad \ldots\ldots (2)$$

The tension in the top of a cylindrical support, or the bottom of a cylindrical load, applied at the level, x, equals

$$C = \pm\frac{W}{2\pi}\tan\theta \quad \ldots\ldots (3)$$

In a closed dome pointed at the top, the stresses near the top are:

$$B = \frac{p\,r^2}{2R\sin\theta}\sec^2\theta - p\,r\tan\theta \quad \ldots\ldots (4)$$

$$A = \frac{-p\,r}{2\sin\theta}\sec\theta \quad \ldots\ldots (5)$$

In a closed dome rounded at the top the stresses are:

$$A = B = -p\frac{R}{2} \quad \ldots\ldots (6)$$

The corresponding equations for pyramoids are:

$$B = \frac{W\,D}{n\,L\,R}\sec^2\theta - p\,D\tan\theta \quad \ldots\ldots (7)$$

$$A = -\frac{W}{n}\sec\theta \quad \ldots\ldots (8)$$

$$C = \pm\frac{W\,D}{n\,L}\tan\theta \quad \ldots\ldots (9)$$

The bending in a horizontal element, 1 ft. high, is,

$$\pm M = \frac{W\,L\,D}{12\,n\,R\,d}\sec^2\theta - p\frac{L^2\,D}{12\,d}\tan\theta \quad \ldots\ldots (10)\ddagger$$

The equations corresponding to Equations (1), (2), and (3), for spherical domes, are:

$$B = \frac{R^2}{r^2}S_0\,p - p\,h \quad \ldots\ldots (11)$$

$$A = -\frac{R^2}{r^2}S_0\,p \quad \ldots\ldots (12)$$

$$C = \frac{R}{r}S_0\,p\,h \quad \ldots\ldots (13)\S$$

* Positive sign indicates tension.

† *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 105, Equations (4), (5), and (6).

‡ *Loc. cit.*, p. 107, Equation (11), corresponding to this formula, is incorrect.

§ *Loc. cit.*, Vol. LV (1905), p. 214, Equation (4).

For conical domes the stresses are:

$$B = -\frac{p\,r^2}{a} \quad \ldots\ldots(14)$$

$$A = -\frac{p\,s^2}{2\,h}\left(\frac{r + r_0}{r}\right) \quad \ldots\ldots(15)^*$$

$$C = \frac{s\,p\,r}{2\,a}(r + r_0) \quad \ldots\ldots(16)$$

For a cone closed at the top:

$$B = -\frac{p\,r^2}{a} \quad \ldots\ldots(17)$$

$$A = -\frac{p\,s^2}{2\,a} \quad \ldots\ldots(18)$$

$$C = \frac{s\,p\,r^2}{2\,a} \quad \ldots\ldots(19)$$

Case 1.—Live Load on One-Half of Dome

Conoidal Formulas.—Assume a homogeneous conoidal dome, with the lines of stress in the center of the thickness of the shell, and with a vertical live load over the right half of its surface (see Fig. 1). The total live load above the level, x, is equal to G, its center of gravity being distant, e, from the axis. The intensity of live load ($= g$ per square foot of surface) is assumed uniform at all points on the same level.

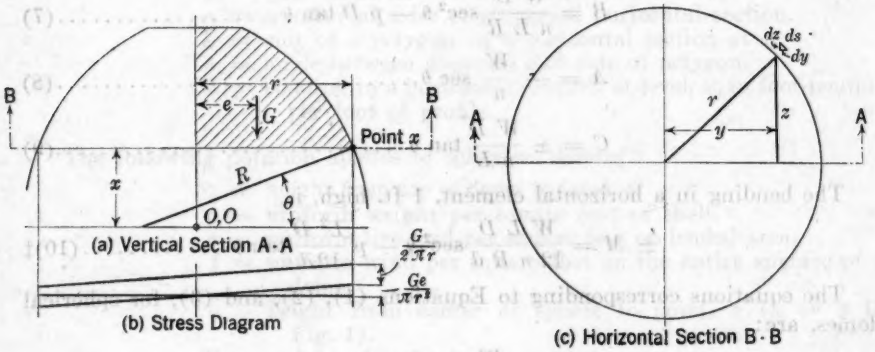

Fig. 1.

On the horizontal section at the level, x, it is roughly assumed that the vertical components of the reactions resisting the unbalanced live load are distributed over the section in accordance with the common theory of flexure; that is, the vertical stress equals a uniform stress due to the load plus or minus a bending stress due to the eccentricity, varying uniformly in proportion to its distance from the neutral axis. It is apparent that the bulging of the dome under load will affect the accuracy of this assumption, but not so seriously as to render the results valueless as a guide.

* *Transactions*, Am. Soc. C. E., Vol. LII (1904), p. 303, Equation (73).

Then, the direct vertical load on the horizontal section is $\frac{G}{2\pi r}$; the moment on the horizontal section is $G\ e$. The section modulus of a cylinder 1 ft. high, about a horizontal axis through the center, is πr^2, derived as follows (see Fig. 2 (a)):

$$\text{Section modulus} = \frac{\pi (d^4 - d_1^4)}{32\ d} = \frac{\pi}{32\ d}(d^2 + d_1^2)(d + d_1)(d - d_1)$$

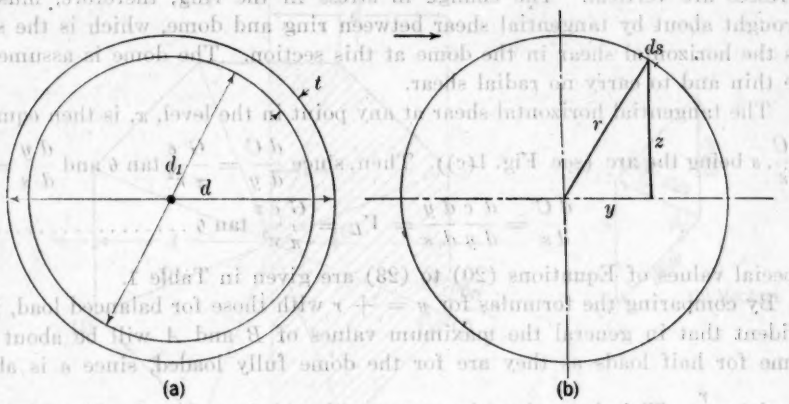

FIG. 2.

Substituting for $d - d_1$, its equivalent, $2\ t$, and if t is small, $d = d_1$, then,

$$\text{Section modulus} = \frac{\pi\ d^2}{4}\ t = \pi\ r^2\ t$$

If d is larger and $t = 1$ the section modulus $= \pi\ r^2$. The extreme fiber stress, in pounds per foot of circumference, due to eccentricity is $\frac{G\ e}{\pi\ r^2}$. The total vertical load at the point, $y = \pm\ r$, in Fig. 1, is $\frac{G}{2\pi r}\left(1 \pm \frac{2\ e}{r}\right)$. The vertical load at any point, y, equals $\frac{G}{2\pi r}\left(1 + \frac{2\ e\ y}{r^2}\right)$. This quantity corresponds with $\frac{W}{2\pi r}$, the uniform vertical component of the reactions due to balanced loads. A similar derivation, therefore, will give the formulas:

$$B_L = \left(1 + \frac{2\ e\ y}{r^2}\right)\left(\frac{G}{2\pi R}\sec^2\theta - \frac{g}{2}\ r \tan\theta\right) \quad \ldots\ldots\ldots\ldots (20)$$

(Note that $\frac{g}{2}$ in Equation (20) corresponds to p in Equation (1), as g covers one-half the section.)

$$A_L = -\left(1 + \frac{2\ e\ y}{r^2}\right)\frac{G}{2\pi r}\sec\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

$$C_L = \pm\left(1 + \frac{2\ e\ y}{r^2}\right)\frac{G}{2\pi}\tan\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

In the case of unbalanced loads there is also a shear on the horizontal and vertical sections at any point; that is, the principal stresses are not horizontal and meridional.

Consider the dome to be cut off at the horizontal section, x, and to rest on the top of a thin cylinder. In general, a ring will be required at the top of the cylinder, and below this ring all the stresses in the cylinder are vertical and varying in amount analogous to any eccentrically loaded pier. The stress in the ring is given by Equation (22), and is seen to vary with y, the distance from the neutral axis. Below the ring there is no horizontal shear, as all stresses are vertical. The change in stress in the ring, therefore, must be brought about by tangential shear between ring and dome, which is the same as the horizontal shear in the dome at this section. The dome is assumed to be thin and to carry no radial shear.

The tangential horizontal shear at any point in the level, x, is then equal to $\frac{d\,C}{d\,s}$, s being the arc (see Fig. 1(c)). Then, since $\frac{d\,C}{d\,y} = \frac{G\,e}{\pi\,r^2}\tan\theta$ and $\frac{d\,y}{d\,s} = \frac{z}{r}$,

$$\frac{d\,C}{d\,s} = \frac{d\,c}{d\,y}\frac{d\,y}{d\,s} = V_L = \frac{G\,e\,z}{\pi\,r^3}\tan\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

Special values of Equations (20) to (23) are given in Table 1.

By comparing the formulas for $y = +\,r$ with those for balanced load, it is evident that in general the maximum values of B and A will be about the same for half loads as they are for the dome fully loaded, since e is about equal to $\frac{r}{2}$. The shear alone is greater with unbalanced load. The values of B_L and A_L are of use in estimating the possible distortion, from which it is possible to judge whether the change in curvature is sufficiently great to be dangerous.

TABLE 1.—Special Values of Equations (20) to (23).

y.	B_L.	A_L.	C_L.	V_L.
$-\,r$	$\left(1-\frac{2\,e}{r}\right)\left(\frac{G}{2\,\pi\,R}\sec^2\theta - \frac{g\,r}{2}\tan\theta\right)$	$-\left(1-\frac{2\,e}{r}\right)\frac{G}{2\,\pi\,r}\sec\theta$	$\pm\left(1-\frac{2\,e}{r}\right)\frac{G}{2\,\pi}\tan\theta$	0
0	$\frac{G}{2\,\pi\,r}\sec^2\theta - \frac{g\,r}{2}\tan\theta$	$-\frac{G}{2\,\pi\,r}\sec\theta$	$\pm\frac{G}{2\,\pi}\tan\theta$	$\frac{G\,e}{\pi\,r^2}\tan\theta$
$+\,r$	$\left(1+\frac{2\,e}{r}\right)\left(\frac{G}{2\,\pi\,R}\sec^2\theta - \frac{g\,r}{2}\tan\theta\right)$	$-\left(1+\frac{2\,e}{r}\right)\frac{G}{2\,\pi\,r}\sec\theta$	$\pm\left(1+\frac{2\,e}{r}\right)\frac{G}{2\,\pi}\tan\theta$	0

Pyramoidal Formulas.—The general formulas for a pyramoidal dome would involve an expression for the section modulus of the polygon about a horizontal axis, which is impracticable. The method may be illustrated by using the most common form of pyramoid—the octagonal; any other form may be solved by a similar process.

Let one-half the dome be loaded, as shown in Fig. 3 (a), the load being G with an eccentricity, e. The value of e may be estimated roughly by eye;

the result will be at least as accurate as the distribution of a chance snow load can ever be. The uniform vertical load on each rib due to G equals $\frac{G}{8}$. The moment equals $G\,e$.

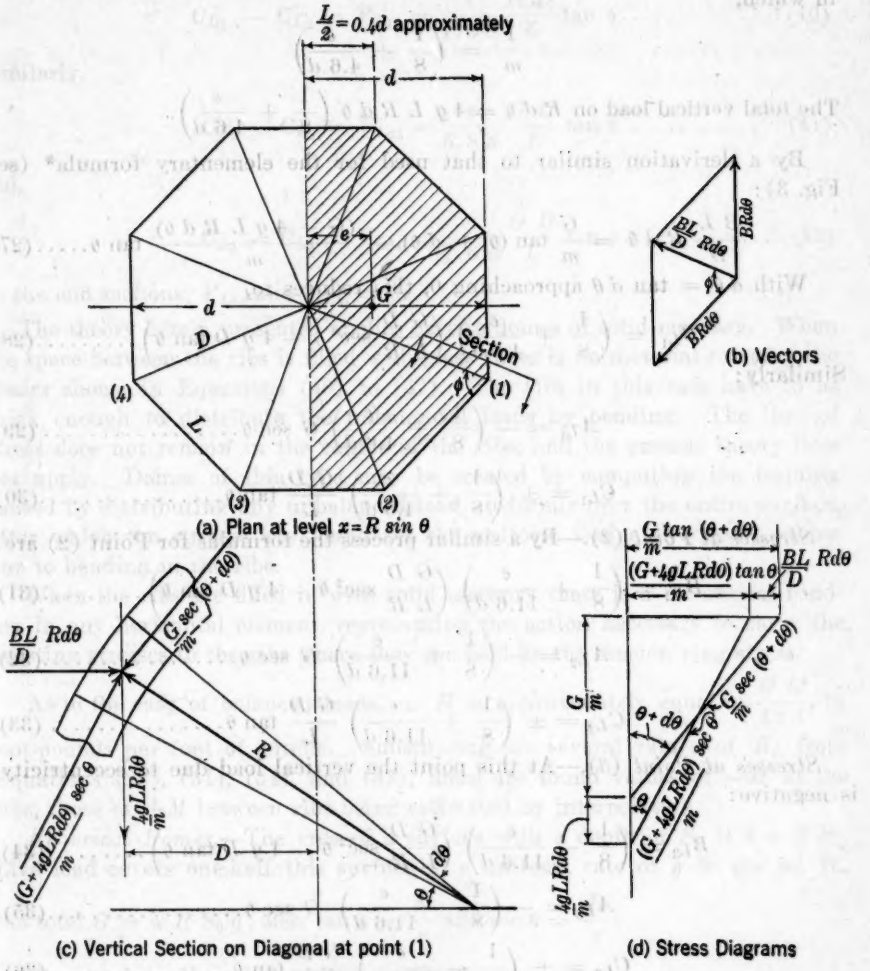

FIG. 3.

Let the vertical loads due to moment be: U_1 and U_2, at Points (1) and (2), respectively; then,

$$4\left(U_1\,d + \frac{U_2\,L}{2}\right) = G\,e \qquad \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

Since U_2 is approximately equal to $0.4\,U_1$ and $\frac{L}{2} = 0.4\,d$, from Equation (24),

$$U_1 = \frac{G\,e}{4.6\,d} \text{ and } U_2 = \frac{G\,e}{11.6\,d} \qquad \ldots\ldots\ldots\ldots\ldots (25)$$

Stresses at Point (1).—The total vertical load above Point (1), (Fig. 3 (*a*)) is:

$$\frac{G}{8} + \frac{G\,e}{4.6\,d} = \frac{G}{m} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

in which,

$$\frac{1}{m} = \left(\frac{1}{8} + \frac{e}{4.6\,d}\right)$$

The total vertical load on $R\,d\,\theta = 4\,g\,L\,R\,d\,\theta\left(\frac{1}{8} + \frac{e}{4.6\,d}\right)$.

By a derivation similar to that used for the elementary formula* (see Fig. 3):

$$\frac{B\,L}{D}\,R\,d\,\theta = \frac{G}{m}\tan(\theta + d\,\theta) - \frac{(G + 4\,g\,L\,R\,d\,\theta)}{m}\tan\theta \quad \ldots\ldots (27)$$

With $d\,\theta = \tan d\,\theta$ approaching 0, this reduces to,

$$B_{L_1} = \left(\frac{1}{8} + \frac{e}{4.6\,d}\right)\left(\frac{G\,D}{L\,R}\sec^2\theta - 4\,g\,D\tan\theta\right) \quad \ldots\ldots\ldots (28)$$

Similarly:

$$A_{L_1} = -\left(\frac{1}{8} + \frac{e}{4.6\,d}\right) G\sec\theta \quad \ldots\ldots\ldots\ldots\ldots (29)$$

$$C_{L_1} = \pm\left(\frac{1}{8} + \frac{e}{4.6\,d}\right)\frac{G\,D}{L}\tan\theta \quad \ldots\ldots\ldots\ldots\ldots (30)$$

Stresses at Point (2).—By a similar process the formulas for Point (2) are:

$$B_{L_2} = \left(\frac{1}{8} + \frac{e}{11.6\,d}\right)\left(\frac{G\,D}{L\,R}\sec^2\theta - 4\,g\,D\tan\theta\right) \quad \ldots\ldots\ldots (31)$$

$$A_{L_2} = -\left(\frac{1}{8} + \frac{e}{11.6\,d}\right) G\sec\theta \quad \ldots\ldots\ldots\ldots\ldots (32)$$

$$C_{L_2} = \pm\left(\frac{1}{8} + \frac{e}{11.6\,d}\right)\frac{G\,D}{L}\tan\theta \quad \ldots\ldots\ldots\ldots\ldots (33)$$

Stresses at Point (3).—At this point the vertical load due to eccentricity is negative:

$$B_{L_3} = \left(\frac{1}{8} - \frac{e}{11.6\,d}\right)\left(\frac{G\,D}{L\,R}\sec^2\theta - 4\,g\,D\tan\theta\right) \quad \ldots\ldots\ldots (34)$$

$$A_{L_3} = -\left(\frac{1}{8} - \frac{e}{11.6\,d}\right) G\sec\theta \quad \ldots\ldots\ldots\ldots\ldots (35)$$

$$C_{L_3} = \pm\left(\frac{1}{8} - \frac{e}{11.6\,d}\right)\frac{G\,D}{L}\tan\theta \quad \ldots\ldots\ldots\ldots\ldots (36)$$

Stresses at Point (4).—

$$B_{L_4} = \left(\frac{1}{8} - \frac{e}{4.6\,d}\right)\left(\frac{G\,D}{L\,R}\sec^2\theta - 4\,g\,D\tan\theta\right) \quad \ldots\ldots\ldots (37)$$

$$A_{L_4} = -\left(\frac{1}{8} - \frac{e}{4.6\,d}\right) G\sec\theta \quad \ldots\ldots\ldots\ldots\ldots (38)$$

$$C_{L_4} = \pm\left(\frac{1}{8} - \frac{e}{4.6\,d}\right)\frac{G\,D}{L}\tan\theta \quad \ldots\ldots\ldots\ldots\ldots (39)$$

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 107, Equation (7).

Shears.—By a process of reasoning similar to that used in regard to conoidal domes, it follows that the horizontal shear in Face (1) to (2) (Fig. 3 (*a*)) is:

$$C_{L_1} - C_{L_2} = V_{L_{12}} = \frac{e}{7.6\,d}\,\frac{G\,D}{L}\tan\theta \quad \ldots\ldots\ldots\ldots (40)$$

Similarly,

$$C_{L_2} - C_{L_3} = V_{L_{23}} = \frac{e}{5.8\,d}\,\frac{G\,D}{L}\tan\theta \quad \ldots\ldots\ldots\ldots (41)$$

and,

$$C_{L_3} - C_{L_4} = V_{L_{34}} = \frac{e}{7.6\,d}\,\frac{G\,D}{L}\tan\theta \quad \ldots\ldots\ldots\ldots (42)$$

In the end sections, $V_{L_{11}}$ and $V_{L_{44}} = 0$.

The theory herein presented applies only to domes of solid masonry. When the space between the ribs is filled with glass there is no material to carry the shears shown in Equations (40) to (42). The ribs in this case have to be thick enough to distribute the unbalanced loads by bending. The line of stress does not remain in the center of the ribs, and the present theory does not apply. Domes of this type may be treated by computing the bending caused by distributing any unbalanced load uniformly over the entire surface, after which the stresses are found for the uniform load and added to those due to bending in the ribs.

When the ribs are filled in with solid masonry there is a theoretical bending in any horizontal element, representing the action necessary to carry the bursting stresses to the ribs where they are held by the tension ring stress.

As in the case of balanced loads, $\pm M$ is approximately equal to $\frac{B\,L^2}{12\,d}$, in foot-pounds per foot of profile. Substituting the several values of B_L from Equations (28), (31), (34), and (37), there are found values of $-M$ at the ribs, those of $+M$ between ribs being estimated by interpolation.

Spherical Domes.—The area of a surface with a depth of S_0 is $2\,\pi\,R\,S_0$. Live load covers one-half this surface at a uniform rate of g lb. per sq. ft., the total $G = \pi\,R\,S_0\,g$; also, $\tan\theta = \frac{h}{r}$ and $\sec\theta = \frac{R}{r}$.

For a sphere Equation (20) becomes,

$$B_L = \left(1 + \frac{2\,e\,y}{r^2}\right)\left(\frac{R^2}{2\,r^2}\,S_0\,g - \frac{g}{2}\,h\right) \quad \ldots\ldots\ldots\ldots (43)$$

Equation (21) becomes,

$$A_L = -\left(1 + \frac{2\,e\,y}{r^2}\right)\frac{R^2}{2\,r^2}\,S_0\,g \quad \ldots\ldots\ldots\ldots (44)$$

Equation (22) becomes,

$$C_L = \pm\left(1 + \frac{2\,e\,y}{r^2}\right)\frac{R}{2\,r}\,S_0\,g\,h \quad \ldots\ldots\ldots\ldots (45)$$

and Equation (23) becomes,

$$V_L = \frac{R}{r^4} S_0 g e z \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (46)$$

The value of e for a complete quarter sphere is $\frac{r}{2}$.

Conical Domes.—The area of surface is $s \pi (r + r_0)$; the live load on one-half this surface is,

$$G = \frac{s \pi g}{2} (r + r_0); \frac{1}{R} = 0; \tan \theta = \frac{r}{a}; \text{ and } \sec \theta = \frac{s}{h}$$

From Equation (20) reduced:

$$B_L = -\left(1 + \frac{2 e y}{r^2}\right) \frac{g r^2}{2 a} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

and,

$$A_L = -\left(1 + \frac{2 e y}{r^2}\right) \frac{g s^2}{4 h} \left(\frac{r + r_0}{r}\right) \ldots\ldots\ldots\ldots\ldots\ldots (48)$$

$$C_L = \left(1 + \frac{2 e y}{r^2}\right) \frac{s g r}{4 a} (r + r_0) \ldots\ldots\ldots\ldots\ldots\ldots (49)$$

$$V_L = \frac{s g e z (r + r_0)}{2 r^2 a} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (50)$$

If the cone is closed at the top, $r = r_0$, $a = h$, and Equations (47) to (50) may be simplified correspondingly.

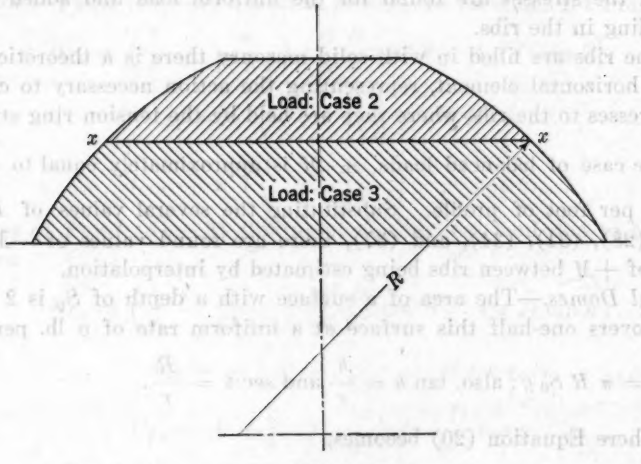

FIG. 4.

CASE 2.—LIVE LOAD IN CENTER OF DOME ONLY

This is shown in Fig. 4, with the load on the area above the level, $x\ x$. Obviously, in the loaded part above $x\ x$, all the stresses are found by the elementary Equations (1) to (25). Below the level, $x\ x$, the formulas are as follows:

$$B_L = \frac{G}{2 \pi R} \sec^2 \theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

This value of B may be greater than that for full load, as the negative term of Equation (1) is missing, p being zero.

$$A_L = -\frac{G}{2\pi r}\sec\theta \quad \ldots\ldots(52)$$

This is necessarily less than for full load, G being less than W. Therefore, it may be neglected.

$$C_L = \frac{G}{2\pi}\tan\theta \quad \ldots\ldots(53)$$

This may be neglected for the same reason as A in Equation (52).

Case 3.—Live Load on Haunch Only

This is shown in Fig. 4, with the load below the line, $x\,x$. Above the level, $x\,x$, the stresses are as determined by the elementary formulas, for dead load only, and all of them are smaller than the stresses for full load, so they may be disregarded. At the top of the loaded area:

$$B_L = -g\,r\tan\theta \quad \ldots\ldots(54)$$

This indicates the fact that when B is compression its maximum value is for a load extending up to the level in question, but not above it. Below the level, $x\,x$,

$$B_L = \frac{G}{2\pi R}\sec^2\theta - g\,r\tan\theta \quad \ldots\ldots(55)$$

which is less, in tension or compression, than the values found by Equations (51) or (54), respectively.

$$A_L = -\frac{G}{2\pi r}\sec\theta \quad \ldots\ldots(56)$$

$$C_L = \frac{G}{2\pi}\tan\theta \quad \ldots\ldots(57)$$

The values of A and C are less than those for full load, and may be neglected.

Case 4.—Wind Stresses

Conoids.—Fig. 5 shows a dome subjected only to a horizontal force, the total of which, above a given section, x, is F. It is assumed for the sake of simplicity, that the effects of wind pressure, friction, and suction are such that every point of the surface is subject to an equal horizontal force, f, per square foot, parallel with the y-axis. The center of pressure is then at the center of gravity of the surface, a distance, e, above the section, x. The wind moment is $F\,e$. The section modulus being $\pi\,r^2$, the extreme fiber stresses are $\pm\frac{F\,e}{\pi\,r^2}$ lb. per ft. of circumference. Similar assumptions are involved as in the case of partial live load.

The vertical component of the stress in the dome at any point in the horizontal section is then $\frac{F\,e\,y}{\pi\,r^3}$. The horizontal force, f, per square foot, parallel to the y-axis, has components: $\frac{f\,y}{r}$, radial, and $\frac{f\,z}{r}$, tangential.

Consider a voussoir (Fig. 6 (*a*)), 1 ft. long and $R\,d\,\theta$ high along the meridion, distance, y, from the neutral axis. The values of r, y, and e are not the same for θ and for $\theta + d\,\theta$, but when $d\,\theta$ approaches zero, the differentials of these values disappear without affecting the result. They are omitted, therefore, in the calculations.

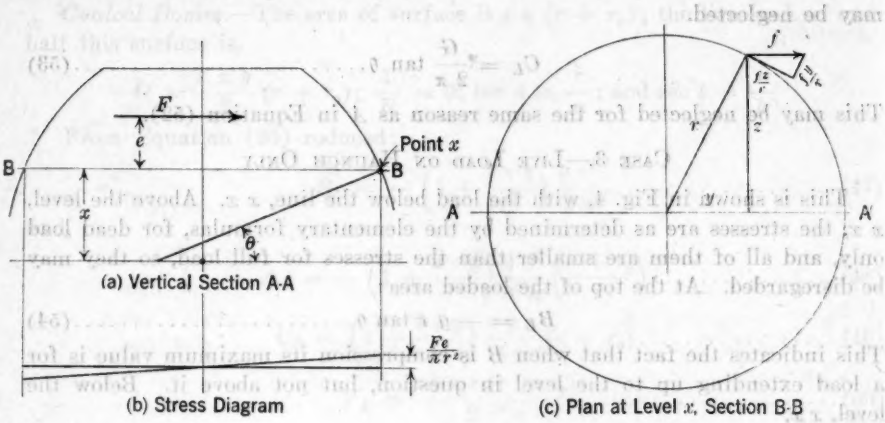

FIG. 5.

The forces acting on the voussoir in the radial plane, are as follows:

1.—Meridional thrust from above, $\frac{F\,e\,y}{\pi\,r^3}\sec(\theta + d\,\theta)$, of which the horizontal component is, $\frac{F\,e\,y}{\pi\,r^3}\tan(\theta + d\,\theta)$, and the vertical component is $\frac{F\,e\,y}{\pi\,r^3}$.

2.—Meridional thrust from below: The total force on the dome down to the bottom level of the voussoir is $F + 2\,\pi\,r\,R\,d\,\theta\,f$. The meridional thrust is $(F + 2\,\pi\,r\,R\,d\,\theta\,f)\frac{e\,y}{\pi\,r^3}\sec\theta$, of which the horizontal component is $(F + 2\,\pi\,r\,R\,d\,\theta\,f)\frac{e\,y}{\pi\,r^3}\tan\theta$, and the vertical component is $(F + 2\,\pi\,r\,R\,d\,\theta\,f)\frac{e\,y}{\pi\,r^3}$.

3.—The horizontal local wind force, f per sq. ft., on the voussoir equals $f\,R\,d\,\theta$ and the component in a radial plane is $f\,R\,d\,\theta\,\frac{y}{r}$.

4.—The resultant of the vertical components of the ring stress, equal to the difference between the vertical components of Force 1 and of Force 2, or $\frac{2\,R\,d\,\theta\,f\,e\,y}{r^2}$.

5.—The resultant of the horizontal components of the ring stress. The value of the ring stress is $B_W\,R\,d\,\theta$.

Let Resultant 5 be called $H\,R\,d\,\theta$. Then, the sum of all the horizontal forces acting in the radial plane may be set equal to zero, thus:

$$H\,R\,d\,\theta + (F + 2\,\pi\,r\,R\,d\,\theta\,f)\,\frac{e\,y}{\pi\,r^3}\tan\theta - \frac{F\,e\,y}{\pi\,r^3}\tan(\theta + d\,\theta) - f\,R\,d\,\theta\,\frac{y}{r} = 0 \quad \ldots\ldots(58)$$

Substituting for $\tan(\theta + d\,\theta)$ its equivalent, $\dfrac{\tan\theta + \tan d\,\theta}{1 - \tan\theta\tan d\,\theta}$, simplifying, and dividing by $\tan d\,\theta = d\,\theta$:

$$H\,R + \frac{\dfrac{2\,R\,f\,e\,y\tan\theta}{r^2} - \dfrac{F\,e\,y}{\pi\,r^3}\tan^2\theta - \dfrac{2\,R\,f\,e\,y}{r^2}\tan^2\theta\tan d\,\theta - \dfrac{F\,e\,y}{\pi\,r^3}}{1 - \tan\theta\tan d\,\theta} - \frac{f\,R\,y}{r} = 0 \quad \ldots\ldots(59)$$

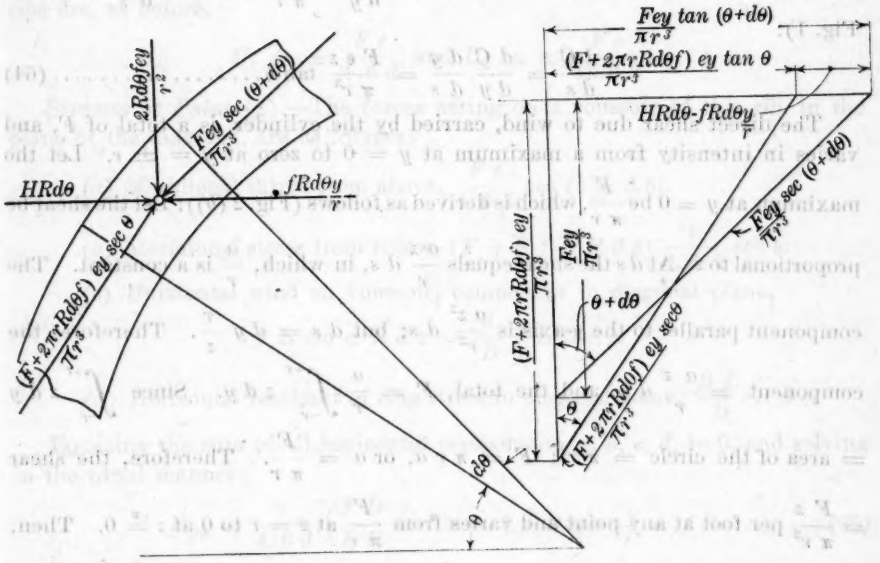

FIG. 6.

Let $d\,\theta = \tan d\,\theta = 0$; then:

$$H = \frac{F\,e\,y}{\pi\,R\,r^3}(1 + \tan^2\theta) - \frac{2\,f\,e\,y}{r^2}\tan\theta + \frac{f\,y}{r} \quad \ldots\ldots(60)$$

The horizontal ring stress, B_W, is equal to $H\,r$:

$$B_W = \frac{F\,e\,y}{\pi\,R\,r^2}(\sec^2\theta) - \frac{2\,f\,e\,y}{r}\tan\theta + f\,y \quad \ldots\ldots(61)$$

From Fig. 6 it is evident that the meridional thrust is:

$$A_W = -\frac{F\,e\,y}{\pi\,r^3}\sec\theta \quad \ldots\ldots(62)$$

Consider the dome to be supported on a thin cylinder, not capable of carrying any radial shear. The wind will be resisted by tangential shears in the cylinder. A ring will be required at the top of the cylinder. The tension in this ring is the horizontal component of the meridional stress, multiplied by r:

$$C_W = \frac{F\,e\,y}{\pi\,r^2} \tan\theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (63)$$

The total shear applied by the dome to the top of the ring at any point is the sum of (1) the tangential shear applied through the ring to the cylinder; and (2) the shear required to produce the change in ring tension occurring at this point. The latter will be analyzed first. Consider an element, $d\,s$, of this tension ring. The change in tension in the elemental distance, $d\,s$, is caused by shear applied by the dome. Then, since $\frac{d\,C}{d\,y} = \frac{F\,e}{\pi\,r^2} \tan\theta$ and $\frac{d\,y}{d\,s} = \frac{z}{r}$ (see Fig. 1):

$$\frac{d\,C}{d\,s} = \frac{d\,C}{d\,y}\,\frac{d\,y}{d\,s} = \frac{F\,e\,z}{\pi\,r^3} \tan\theta \ldots\ldots\ldots\ldots\ldots\ldots (64)$$

The direct shear due to wind, carried by the cylinder, is a total of F, and varies in intensity from a maximum at $y = 0$ to zero at $y = \pm r$. Let the maximum at $y = 0$ be $\frac{F}{\pi\,r}$, which is derived as follows (Fig. 2 (*b*)): Let the shear be proportional to z. At $d\,s$ the shear equals $\frac{a\,z}{r}\,d\,s$, in which, $\frac{a}{r}$ is a constant. The component parallel to the y-axis is $\frac{a\,z^2}{r^2}\,d\,s$; but $d\,s = d\,y\,\frac{r}{z}$. Therefore, the component $= \frac{a\,z}{r}\,d\,y$, and the total, $F = \frac{a}{r}\int_{-r}^{+r} z\,d\,y$. Since $\int_{-r}^{+r} z\,d\,y$ = area of the circle $= \pi\,r^2$; $F = \pi\,r\,a$, or $a = \frac{F}{\pi\,r}$. Therefore, the shear $= \frac{F\,z}{\pi\,r^2}$ per foot at any point and varies from $\frac{F}{\pi\,r}$ at $z = r$ to 0 at $z = 0$. Then, the shear at any point will be about $\frac{F}{\pi\,r}\,\frac{z}{r} = \frac{F\,z}{\pi\,r^2}$, of which the component parallel to the wind is $\frac{F\,z^2}{\pi\,r^3}$. The sum of these components is,

$$\int_{-r}^{+r} \frac{F\,z^2}{\pi\,r^3}\,d\,s = \int_{-r}^{+r} \frac{F\,z}{\pi\,r^2}\,d\,y$$

Now, $\int_{-r}^{+r} z\,d\,y$ is the area of the circle $= \pi\,r^2$; so the sum of components parallel to F is $\frac{F}{\pi\,r^2} \times \pi\,r^2 = F$, as required.

The ring tension is greatest at the leeward side, so the tendency of the shear between the dome and the ring is, in a sense, to drag the ring toward the left. The tendency of the shear between the dome and the ring due to the

direct wind thrust is, in a sense, to drag the ring toward the right. Therefore, the total horizontal shear at any point is the difference of these two shears, or:

$$V_W = \frac{F\,e\,z}{\pi\,r^3}\tan\theta - \frac{F\,z}{\pi\,r^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (65)$$

At the neutral axis, where $z = r$, this shear is a maximum of $\frac{F\,e}{\pi\,r^2}\tan\theta - \frac{F}{\pi\,r}$. At the ends of the y-axis, the shear is 0 since z is 0.

Pyramoids.—As in the case of unbalanced live load, the method will be illustrated with the octagonal pyramoid shown in Fig. 3. Consider it to be acted upon by a uniform horizontal force of f per sq. ft. over its entire surface, making a total shear, F, with an eccentricity, e.

The wind moment is $F\,e$, and the vertical loads due to this moment on the ribs are, as before,

$$U_1 = \pm\frac{F\,e}{4.6\,d}, \text{ and } U_2 = \pm\frac{F\,e}{11.6\,d}$$

Stresses at Point (1).—The forces acting on a voussoir of this rib, in the plane of the diagonal, are as follows:

(*a*) Meridional thrust from above, $\frac{F\,e}{4.6\,d}\sec(\theta + d\,\theta)$.

(*b*) Meridional stress from below, $(F + 8\,f\,L\,R\,d\,\theta)\frac{e}{4.6\,d}\sec\theta$.

(*c*) Horizontal wind on voussoir, component in diagonal plane,

$$f\,L\sin\phi\,R\,d\,\theta = \frac{d}{D}f\,L\,R\,d\,\theta.$$

(*d*) Horizontal resultant of ring stress in diagonal plane, $\frac{B\,L}{D}R\,d\,\theta$.

Equating the sum of all horizontal components, *a*, *b*, *c*, *d*, to 0, and solving in the usual manner:

$$B_{W1} = \frac{e}{4.6\,d}\left(\frac{F\,D}{L\,R}\sec^2\theta - 8\,f\,D\tan\theta\right) + f\,d \quad \ldots\ldots (66)$$

Similarly,

$$A_{W1} = -\frac{e}{4.6\,d}F\sec\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots (67)$$

$$C_{W1} = \frac{1}{2}\,\frac{F\,e}{4.6\,d}\tan\theta\sec\phi;\ \sec\phi = \frac{2\,D}{L} \quad \ldots\ldots\ldots\ldots (68)$$

$$C_{W1} = \pm\frac{e}{4.6\,d}\,\frac{F\,D}{L}\tan\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots (69)$$

Stresses at Point (2).—By analogy:

$$B_{W2} = \frac{e}{11.6\,d}\left(\frac{F\,D}{L\,R}\sec^2\theta - 8\,f\,D\tan\theta\right) + \frac{f\,L}{2} \quad \ldots\ldots (70)$$

$$A_{W2} = -\frac{e}{11.6\,d}F\sec\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots (71)$$

$$C_{W2} = \pm\frac{e}{11.6\,d}\,\frac{F\,D}{L}\tan\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots (72)$$

Stresses at Point (3):

$$B_{W3} = -\frac{e}{11.6\,d}\left(\frac{F\,D}{L\,R}\sec^2\theta - 8\,f\,D\tan\theta\right) - \frac{f\,L}{2} \quad \ldots\ldots (73)$$

$$A_{W3} = +\frac{e}{11.6\,d}\,F\sec\theta \quad \ldots\ldots (74)$$

$$C_{W3} = \mp\frac{e}{11.6\,d}\,\frac{F\,D}{L}\tan\theta \quad \ldots\ldots (75)$$

Stresses at Point (4):

$$B_{W4} = -\frac{e}{4.6\,d}\left(\frac{F\,D}{L\,R}\sec^2\theta - 8\,f\,D\tan\theta\right) - f\,d \quad \ldots\ldots (76)$$

$$A_{W4} = +\frac{e}{4.6\,d}\,F\sec\theta \quad \ldots\ldots (77)$$

$$C_{W4} = \mp\frac{e}{11.6\,d}\,\frac{F\,D}{L}\tan\theta \quad \ldots\ldots (78)$$

Shears include the change in C_W minus the direct wind shear, as follows:

$$V_{W12} = \frac{e}{7.6\,d}\,\frac{F\,D}{L}\tan\theta - 0.15\,F, \text{ approximately} \quad \ldots\ldots (79)$$

$$V_{W23} = \frac{e}{5.8\,d}\,\frac{F\,D}{L}\tan\theta - 0.21\,F, \text{ approximately} \quad \ldots\ldots (80)$$

The moment in a horizontal element of the face equals $\pm M$ (approximately) $= \frac{B\,L^2}{12\,d}$ due to thrusts. To this is added the moment due to wind directly applied: For Line (1) (1) and Line (4) (4) (Fig. 3) $M = \frac{f\,L^2}{12}$; for Line (1) (2) and Line (3) (4) $M = \frac{f\,L^2}{12\sqrt{2}}$; and for Line (2) (3), $M = 0$.

The stresses due to these moments are to be added to the ring stresses found from Equations (66), etc.

Wind, spherical domes, etc., formulas analogous to Equations (43), etc., can be derived by substituting the factors for the sphere, cone, etc.

Maximum Stresses

It is possible, by the use of the foregoing equations, to obtain for any point in a dome the combined horizontal ring stresses, meridional thrusts, and shears on the horizontal plane, for dead, live, and wind loads. Only in extreme cases, such as unprecedented size or shape, would it be necessary, of course, to go into such detailed investigation. Supposing it to be desirable in a given case, having found the total stresses, B_T, A_T, C_T, and V_T, it remains to find the maximum or principal stresses, and the direction in which they lie. The following method is derived from Rankine.

Consider an elementary surface, $o\,a\,b$ (Fig. 7), with its sides perpendicular to A_T, B_T, and the principal stress, A_P. A_T and B_T are positive as tension in the directions, $A\,o$ and $B\,o$, the balancing stress, A_P, is positive as tension in the direction, $o\,e = o\,f$. The shear, V_T, is positive as clockwise on the vertical plane, and is, therefore, opposite in sign to the direct stresses, A_T and B_T.

Let $d\,o$ and $c\,o$ be the total components acting on the element parallel to the axes; their balancing force is the same as $o\,f$, the normal force, since there is no tangential force in a principal plane.

$$c\,o = A_T\,o\,b - V_T\,o\,a = A_T\,a\,b\cos\alpha - V_T\,a\,b\sin\alpha$$
$$d\,o = B_T\,o\,a - V_T\,o\,b = B_T\,a\,b\sin\alpha - V_T\,a\,b\cos\alpha$$
$$o\,f = d\,o\sin\alpha + c\,o\cos\alpha$$
$$f\,e = c\,o\sin\alpha - d\,o\cos\alpha$$

$$A_P = \frac{o\,f}{a\,b} = B_T\sin^2\alpha - V_T\sin\alpha\cos\alpha + A_T\cos^2\alpha - V_T\sin\alpha\cos\alpha$$
$$= A_T\cos^2\alpha + B_T\sin^2\alpha - 2\,V_1\sin\alpha\cos\alpha \quad \ldots\ldots\ldots\ldots (81)$$

$$V_P = O = \frac{f\,e}{a\,b} = A_T\cos\alpha\sin\alpha - V_T\sin^2\alpha - B_T\cos\alpha\sin\alpha$$
$$+ V_T\cos^2\alpha = O \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (82)$$

from which,

$$\tan 2\,\alpha = \frac{-2\,V_T}{A_T - B_T} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (83)$$

A similar demonstration will show that:

$$B_P = A_T\sin^2\alpha + B_T\cos^2\alpha + 2\,V_T\cos\alpha\sin\alpha \ldots\ldots\ldots (84)$$

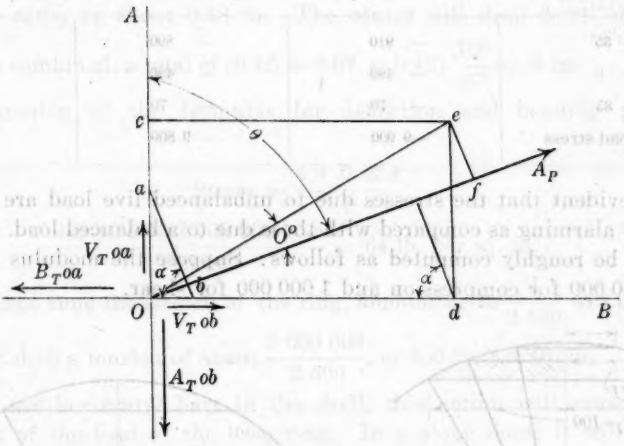

FIG. 7.

EXAMPLES

The foregoing theory may be illustrated by the following examples.

Example 1.—Unbalanced Load.—Consider the dome shown in Fig. 8 having a spherical surface of radius, 200 ft., and $r = 100$ ft.; let the dead load consist of 8-in. concrete weighing 100 lb. per sq. ft., and the live load, 10 lb. per sq. ft. on the right half only.

In Table 2, stresses are given for various values of θ, r, and e. (See Fig. 8.) They were determined by substituting the given data* in Equations (20), (21), and (23).

* The data given are approximate.

TABLE 2.—STRESSES DUE TO UNBALANCED LIVE LOAD.

y.	B.	A.	V.
$\tan\theta_1 = 1.74$; $\sec\theta_1 = 2$; $r = 100$ ft.; $e = 45$ ft.; $G = 170\,000$ lb.; and area of dome = 34 000 sq. ft.			
+100	— 630	— 1 030	0
0	— 330	— 540	425
—100	— 30	— 50	0
Dead load stress	—6 600	—10 800	0
$\tan\theta_2 = 2.65$; $\sec\theta_2 = 2.82$; $\sec^2\theta_2 = 7.95$; $r = 71$ ft.; $e = 31$ ft.; area of dome above this level = 16 400 sq. ft.; and $G = 80\,000$ lb.			
+ 71	— 800	— 960	0
0	— 430	— 510	420
— 71	— 60	— 70	0
Dead load stress	—8 300	—10 400	0
$\tan\theta_3 = 5.6$; $\sec\theta_3 = 5.7$; $\sec^2\theta_3 = 32.5$; $r = 35$ ft.; $e = 15$ ft.; area = 3 700 sq. ft.; and $G = 18\,500$ lb.			
+ 35	— 910	— 890	0
0	— 490	— 480	400
— 35	— 70	— 70	0
Dead load stress	—9 900	— 9 800	....

It is evident that the stresses due to unbalanced live load are not, in this case, very alarming as compared with those due to a balanced load. The distortion may be roughly computed as follows: Suppose the modulus of elasticity to be 2 000 000 for compression and 1 000 000 for shear.

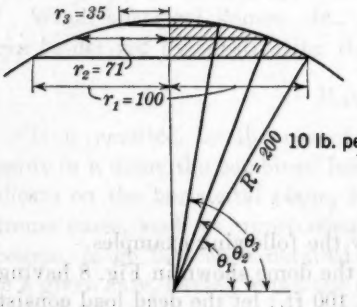

FIG. 8.

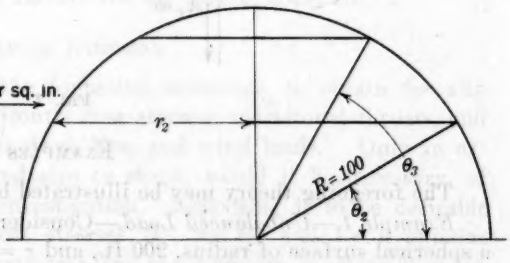

FIG. 9.

Shear Distortion of the Neutral Axis from the Base Up to the Level of θ_2.— There is a stress of 400 lb. in 8 in. by 12 in., or 4 lb. per sq. in. The section is about 400 in. high along the profile. The distortion equals $\frac{4 \times 400}{1\,000\,000} = 0.0015$ in.

Compression Distortion of the Horizontal Ring at the Level, θ_2.—The right half is compressed under an average stress of about 600 lb. per ft. of height, or 6 lb. per sq. in. of concrete; its length is 2 640 in. The total strain on the right half is $\frac{6 \times 2\,640}{2\,000\,000} = 0.008$ in. Similarly, the left side is strained; $\frac{2.5 \times 2\,640}{2\,000\,000} = 0.003$ in.

Therefore, the right half circle is about 0.005 in. smaller in arc length than the left, and at the same time has been moved toward the left about 0.00015 in. It is evident that these distortions are not alarming.

The distortions due to balanced loads and temperature may be treated here by a similar approximate method. If the dome and the ring at the base change their temperature equally, the distortion will obviously be slight. A temperature drop of 100° Fahr. on the dome only, would shorten the half profile $\frac{1}{1\,500}$, or about 0.85 in., relative to the ring. The average meridional dead load thrust is about 105 lb. per sq. in. Using a modulus of elasticity of 2 000 000, this gives a shortening of the half profile of about 0.07 in. The ring, if designed for 12 000 lb., will lengthen 1 part in 2 500, and r will increase in the same ratio, or about 0.48 in. The center will drop down, due to all these factors combined, a total of $(0.85 + 0.07 + 0.48)\frac{100}{27} = 5$ in.

A combination of the formulas for deflection and bending gives the formula

$$\text{Stress} = \frac{4.8\,D\,E\,d}{l^2}$$

$$\frac{4.8 \times 5 \times 2\,000\,000 \times 8}{5\,760\,000} = 64 \text{ lb. per sq. in.}$$

At the same time the stretch of the ring, amounting to $\frac{1}{2\,500}$ will impose on the adjacent shell a tension of about $\frac{2\,000\,000}{2\,500}$, or 800 lb. per sq. in.

If there are horizontal bars in the shell, this action will cause them to assume part of the load of the base ring. In a stone dome it will open the vertical joints to a point where the dome will settle enough to keep the joints closed. Between this point and the ring the shell will have a conical surface of stress, tangent to the theoretical surface, and running to about $\frac{1}{2}$ in. outside the theoretical point of support.

Example 2.—Consider a hemi-spherical dome of 100 ft. radius, built of 8-in. concrete weighing 100 lb. per sq. ft., and with a wind of 10 lb. per sq. ft. of superficial area. (See Fig. 9.) The formulas used are Equations (61), (62), and (65), and the results are arranged for convenience in Table 3.

Distortion.—Shear Deflection of the Neutral Axis Up to the Level of θ_2.—The average shear $\left(\frac{1\,500}{8 \times 12}\right) = 15$ lb. per sq. in. The length of the arc is 660 in. The deflection equals $\frac{15 \times 660}{1\,000\,000} = 0.01$ in.

Tension Distortion in Right Half of Ring at Level of θ_2.—The average tension is 480 lb. on a section 8 by 12 in., or 5 lb. per sq. in. The length of the half ring is 3 250 in. The strain $= \frac{5 \times 3\,250}{2\,000\,000} = 0.008$ in. expansion.

Similarly, the left side has about 0.008 in. of contraction. The right semicircle is then about 0.016 in. longer than the left one, and, at the same time, has been moved toward the right about 0.01 in. by shear deflection.

TABLE 3.—STRESSES DUE TO WIND LOAD.

y.	B_W.	A_W.	V_W.
$\tan\theta_1 = 0$; $\sec\theta_1 = 1$; $r = 100$ ft.; $e = 50$ ft.; $R = 100$ ft.; area = 63 000 sq. ft.; and $F = 630\,000$ lb.			
+100	2 000	— 1 000	0
0	0	0	— 2 000
—100	— 2 000	+ 1 000	0
Dead load stress	+10 000	—10 000	0
$\tan\theta_2 = 0.57$; $\sec\theta_2 = 1.15$; $R = 100$ ft.; $r = 87$ ft.; $e = 25$ ft.; area = 31 400 sq. ft.; and $F = 314\,000$ lb.			
+87	970	— 380	0
0	0	0	— 970
—87	— 970	+ 380	0
Dead load stress	+1 700	—6 700	0
$\tan\theta_3 = 1.74$.; $\sec\theta_3 = 2$; $R = 100$ ft.; $r = 50$ ft.; $e = 7$ ft.; area = 8 800 sq. ft.; and $F = 88\,000$ lb.			
50	410	— 160	0
0	0	0	— 430
—50	— 410	+ 160	0
Dead load stress	—3 000	—5 600	0

RELATION TO THE ELASTIC THEORY

The exact theory of domes, as distinguished from the common theory as given in this and the preceding paper, involves the elastic deformations caused by the stresses, and their effect on the stresses themselves. A treatment of the elastic theory as applied to conoids with balanced loads may be found in a paper by Dr. Peter Pasternak,* of Zurich, Switzerland.

It is obvious that the elastic theory is of great value as a means of understanding the nature of the secondary stresses as well as for finding their exact amount. The common theory can be used with assurance only in those cases where the distortions are too small to be dangerous. The examples

* "Die practische Berechnung biegefester Kugelschalen, kreisrunder Fundamentplatten auf elastischer Bettung und kreiszylindrischer Wandungen in gegenseitiger monolither Verbindung," *Zeitschrift* für angewandte Mathematik und Mechanik, Band 6, 1926.

indicate a method of judging whether the dome can be designed safely by the common theory, or whether it must be studied by the more elaborate elastic formulas.

Summary

In this paper the theory covering symmetrical loads and stresses for domes is extended to include formulas for vertical loads occupying one side of the dome only, the area above a given level, or the area below a given level. There is also a set of formulas for wind stresses.

These give stresses which are in most cases necessarily less than those for symmetrical loads, or else they may constitute so small an addition as to be negligible in practice. Their value is chiefly in providing a means by which the engineer who is faced with a dome of unusual size or shape may assure himself in regard to the order of magnitude of the neglected stresses.

These formulas, taken in this way as safeguards rather than as "exact" equations, constitute a complete practical theory for domes of all kinds having a homogeneous surface and a polygonal or circular plan.

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THE PLANNING OF CAPITAL CITIES: DENVER, COLORADO*

By S. R. DeBoer,† Esq.

Synopsis

As this is the first of an intended series of papers on the "Planning of Capital Cities", some general remarks may serve to define their scope.

Capital cities dot the map of the United States from ocean to ocean and from Mexico to Canada. They include varying conditions of climate, commerce, and physical characteristics; but they are alike in that each is the seat of government of a Sovereign State. Some are old, others are relatively new and are still developing "growing pains". Some are located on ocean, lake, or river, some enjoy the clear vision of mountain slope or range. A few are located on outstretching plains, and many are favored with an eminence on which to place their significant building which is the emblem of State authority.

It is always of interest in studying the origin and development of an important city to know who made the first street plan; how this has been enlarged from time to time and whether the city has a planning commission and has been zoned. Because of the many lessons that can be drawn, and the older history of development which can be studied, reference is made in the paper to several of the better known European capitals. The City Planning Engineer can find much of interest to study in foreign capitals.

After noting the general characteristics of capital cities, the paper presents a description of Denver, the capital of Colorado, and outlines its early development and transitions in stages of growth. Attention is called to the various attempts in city planning; to the fact that the location of the Capitol has acted as a nucleus and, to some extent, has dominated the street system.

Considerable attention is given to the recreational system of Denver and its environs, as to the application of principles in the development of the Denver plan, and the unique features of zoning which are applied thereto.

Note.—Written discussion on this paper will be closed in December, 1929.

* Presented at the Technical Session, Denver, Colo., July 13, 1927.

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General Characteristics of Capital Cities

A city may be termed the "service station" of its district. It performs the work which the surrounding region requires. Thus, based on their means of existence, cities can be classified as factory, mining, agricultural, commercial, residential, or tourist cities; health resorts or educational centers; and military or capital—that is, governmental—cities. It is true that most urban communities include more than one of these classifications, and large cities often cover all of them; but in the life of each, except the metropolitan center, some one feature is predominant. In the United States, few capital cities have a metropolitan character. In planning for their future, it is proper, therefore, that major emphasis be placed on those features which will focus attention on the seat of the Government.

It should be remembered that, not only are the reins of government centered in a capital city, but also the pride of a State or of a Nation. Modern civilization is not as yet far enough removed from the feudal days, but that the pomp and splendor attending governmental functions are still objects of admiration. This interest, however, is shifting from individuals to buildings and cities. Democracy has taken the diamond-studded uniform and sword from the rulers of this country, but its citizens still want to point with pride to their government, and this pride is more and more becoming embodied in buildings and cities.

These fundamental facts determine the character of the plan according to which a government city should be built. It must be laid out on liberal lines, with wide boulevards and attractive parks. It must have large, open plazas, around which imposing governmental buildings can be grouped, and the main group with its plaza should be developed into a civic center.

Foreign Capitals

The civic-center idea reaches far back into the history of Rome when the Emperors beautified large areas for the city, mostly for their own glorification; but the idea of beautifying capital cities and planning them from the outset had its origin with Louis XIV, the self-styled "King of the Sun", of France. He hired Le Notre to plan the gardens and city of Versailles on a scale of magnificence which the world, at that date, had not witnessed. Jealous of his fame, other princes began to build similar cities, or to beautify their existing ones. A German prince immediately began the building of Karlsruhe. Czar Peter the First built St. Petersburg, and then other cities—Vienna, Berlin, and Paris—adopted extensive improvements.

The City of Paris had corrective and beautifying plans as far back as 1664. It seems that the Royal House of France retarded the development of these plans, fearing the City would become too powerful. The first basic plan was developed after the Revolution, and it was partly carried out by the First Napoleon as a military necessity in controlling the city. The Third Napoleon, with the assistance of Haussmann, completed more of the work, also for military purposes. In spite of the military influence, Paris has developed a plan that is following the general lines characteristic of a capital city, with

the exception of the location of railroad stations, which have no particular relation to the center of the city.

The City of Vienna is an interesting illustration of a capital developed around an existing old city. The "Ring Street", built on the old fortifications, has become the location of the Government buildings and the royal palaces. The outer streets radiate from this circle boulevard. The inner part of Berlin is carefully planned with radiating traffic lines and circular boulevards, which are placed mainly on the old fortifications. The newer parts of Berlin are not as well planned as the older part.

The City of London has no apparent design. It has been built on the principle of expediency of the moment rather than by following a comprehensive idea. In spite of this, the Houses of Parliament, Buckingham Palace, and the Administrative Offices along Whitehall are more or less connected. The remainder of the city seems to follow the lines of cowpaths rather than planned city streets. Through the conglomeration of blocks, major streets wind their more or less indirect ways. The proposal of the great Christopher Wren would have made London a well-planned city after the fire in 1666; but it was never adopted. The reason London can exist and grow, in spite of this street handicap, is the fact that its underground transit system supplies the need for arterial and circular lines.

Karlsruhe, Germany, represents a good illustration of a city built along radiating lines. All streets are focused on the governmental center—in this case, the palace of the ruling prince. Two-thirds of the circle is occupied by the Royal Park placed back of the palace. The city in front is well planned, with open plazas in various places. Triangular plots, caused by the intersection of radiating lines, are carefully avoided.

All the plans thus far discussed were designed before modern means of transportation affected the city plan. The plan of Canberra, Capital City of Australia, takes into consideration not only this, but also the still newer ideas of zoning. Each of several centers is laid out as an independent city, connected with the other units by broad and direct lines of traffic. Canberra has a main civic center—the group of Government Buildings. Around it are grouped secondary centers, such as the municipal, business, residential, industrial, and the suburban districts.

The Capital of Colorado

All these cities are capitals of large nations, in which the Government center is dominant. In the forty-eight State capital cities of this country, the governmental feature is of less consequence. Many of them, however, are typically administrative cities, and would not be what they are to-day were it not for this activity.

The City of Denver is an outstanding example of a capital city in America. It is third in population among State capital cities. Built at the foot of the Rockies (Fig. 1), it has as its background 200 miles of the Rocky Mountain range, towering up to peaks more than 14 000 ft. high. As seen from the city, this affords one of the most magnificent vistas any capital ever had, but, at the same time, it has been one of the most serious barriers to commerce. Denver had its beginning as a mining town in 1858, and immediately became the lead-

ing city of a large district. Many of the mountain valleys open into the plains near the city, and it soon became the distributing point to the mining cities in the mountains.

When the first transcontinental railroad was built by the Union Pacific Company, the proposed line over Berthoud Pass was abandoned as impractical, and the line was built through Southern Wyoming, with Cheyenne as a division point. It was a hard blow to the small city of Denver, and it is said that its population decreased to one-half. Many cities are aided rather than harmed by serious calamities; the fight and co-operation necessary to overcome them may carry the city to far greater success than it would have reached otherwise. The citizens of Denver were jarred into action by this location of a railroad that ignored their city. They built a connecting line from Denver to Cheyenne. When they realized that they actually could do such things, it took but a few years for them to build other lines, and, thus, at the time Colorado was admitted to Statehood, the city was the center of eight railroads.

The Rocky Mountains have always been a barrier against the city's railroad connections to the West. At present, Denver has thirteen railroad lines reaching out in all directions, with three minor lines reaching into the mountain region. In 1907 a railroad was built over the mountain range, with a maximum altitude of nearly 11 000 ft. This road has wonderful scenic advantages, but transportation during the winter is almost impossible. To avoid the heavy grade over the range, the Moffat Tunnel six miles in length has been built through the mountains. It was financed by the counties served, Denver carrying the major part. This tunnel will eventually put Denver on a major transcontinental line. Connecting lines west of the tunnel are still to be built, but once this main line is in operation, the distance to the Pacific Coast and to Salt Lake City, Utah, will be reduced by 250 miles.

Being located in the arid region, its water supply is another problem of the City of Denver. It has ample water for domestic use at present, but with the expected growth, the city is laying plans for a far greater supply. The quantity of water contained in the drainage areas of the State is limited, and a large part of it must be used for the irrigation of farm lands. The city cannot utilize this irrigation water because, with the growth of the municipality, the watering of a larger acreage of farm land around the city will become as essential as the greater domestic supply. Scientific use of water will be one of the problems the city must face. Plans have been effected to use the pioneer bore of the Moffat Tunnel for an additional supply of water to be carried from the western slope of the mountains to within reach of the city.

A bluff overlooking the city was selected for the site of the State Capitol. Three blocks of ground were acquired, sloping up from the city's main thoroughfare to the top of this hill on which the Capitol was built. The building was placed in such a way that it commands a beautiful view of the Rockies (Fig. 1). The tower is located on the axis of the city's main shopping street, but the building is shut off from this street by one block of stores. In the early days, the city failed to open this shopping street to the Capitol grounds and since that time such efforts have been unsuccessful.

Fifteen years after the completion of the State Capitol, the city prepared plans for the development of a civic center, as a continuation of the Capitol

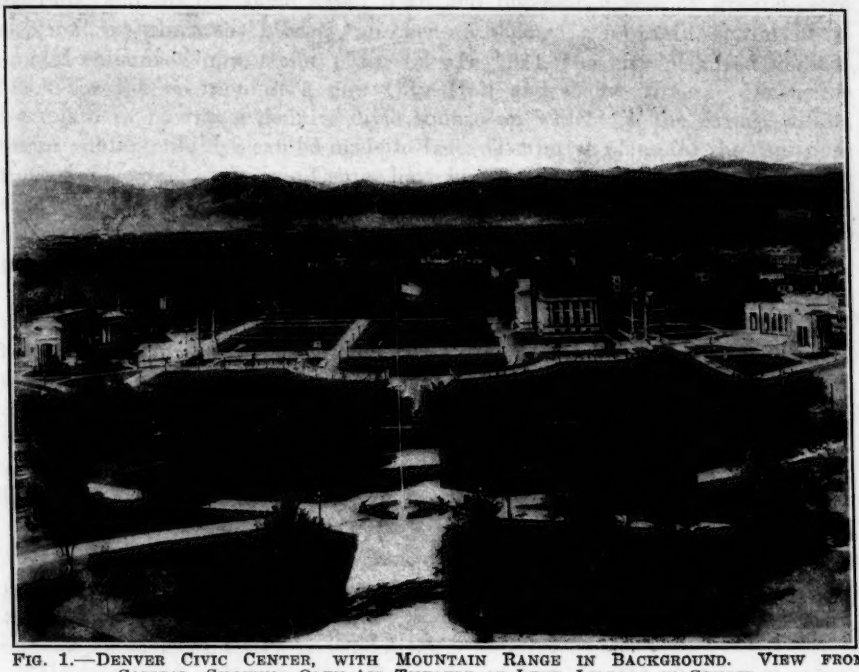

FIG. 1.—DENVER CIVIC CENTER, WITH MOUNTAIN RANGE IN BACKGROUND. VIEW FROM CAPITOL, SHOWING OPEN-AIR THEATRE AT LEFT, LIBRARY IN CENTER, AND COLONNADE AT RIGHT.

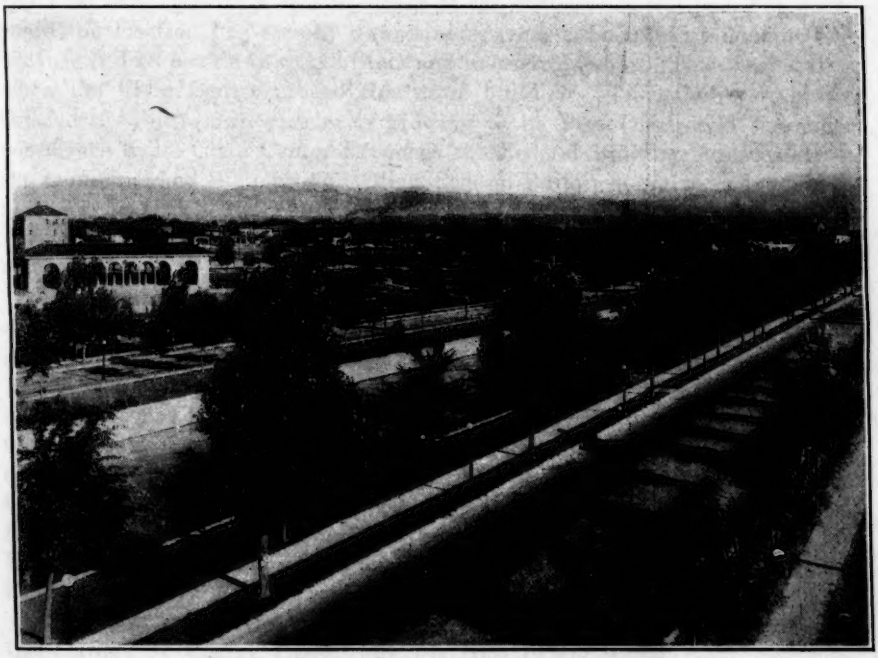

FIG. 2.—VIEW OF SPEER BOULEVARD, DENVER, COLO.

grounds, and on the same axis. Two and one-half blocks of ground were acquired, on which the Library, an open-air theatre, a fountain, and an ornamental colonnade now stand (Fig. 1). In 1924, the city acquired another block for the erection of a new City Hall and Court House. Denver is fortunate in having a definite civic square on which, in the future, all its major public buildings can be made to face. Tentative plans for the continuation of this State and Civic Center have been made, and there is no doubt but that, eventually, this park will be more than ¾ mile in length. The Federal Mint is already located on this extension, and an art gallery will very likely follow.

Denver is still an infant among cities. It is scarcely more than threescore years old, but it may be stated that it has sensed its calling as a capital better than many an historical city. At a considerable expense, it has acquired this civic center in front of the State Capitol, and has begun laying it out in one of the best known, if not the most attractive, municipal centers of the world. This has proved to be a good investment. The actual return is an indirect one, and is difficult to estimate, but it is said that the city's added income pays for the cost of its Civic Center every two years.

Regardless of the accuracy of these figures, its value is shown in the attitude of the citizens themselves. From active opposition during the period of construction, this viewpoint has changed to one of impatience for any delays in continuing the work, and of anxiously guarding the treasure against mistakes of design or vandalism.

Denver is planning another Government center. The Federal Building and Post Office was in a remote corner of the commercial section. For many years the location has seemed unfortunate, from a business standpoint and because it does not tie in to the State and municipal group. There was a time when the Civic Center needed this extra building. Time, however, healed this, also. Down-town Denver is growing to its Post Office, and the added confidence in its Civic Center has made the Federal Building unnecessary to the municipal and State group. Other Federal Buildings are needed, however, and now the City contemplates a special plaza for this purpose. The people of Denver would not think of a second center had its main Civic Center not been a success.

The population curve of Denver shows a steady growth of 2.1% annually. At the present rate, the city will have a population of 520 000 in 1950, and plans are now being completed for a major street layout based on this estimate. Traffic studies in the down-town district have shown conclusively the need of a belt boulevard for traffic around the business district. This line will consist of wide boulevards that will tie the municipal and State centers to the Government center, to the hotel district, to the Union Station, and to the major arterial lines. It will encircle the business district, and traffic can enter from all four sides. This belt boulevard will take on very much the character of the Ringstrasse of Vienna or Cologne and, when completed, will be monumental in its appearance. From it radiate all the arteries of the city.

The main radial line of the city is Speer Boulevard (see Fig. 2) built on the borders of Cherry Creek. At the time of its construction, the creek bottoms were the dumping grounds for the city. The Boulevard was much

opposed, but its value is now shown in the 26 000 cars that pass over it daily. As an attractive feature in the Denver plan, it is just as important. The continuation of the Boulevard north and south is one of the proposed lines of the present plan of Denver (see Fig. 3). The other major traffic boulevard that will become part of the belt is the present 20th Street. Its widening and extension will carry it to the northern entrance of the city, and south to a line that reaches to the southern limits.

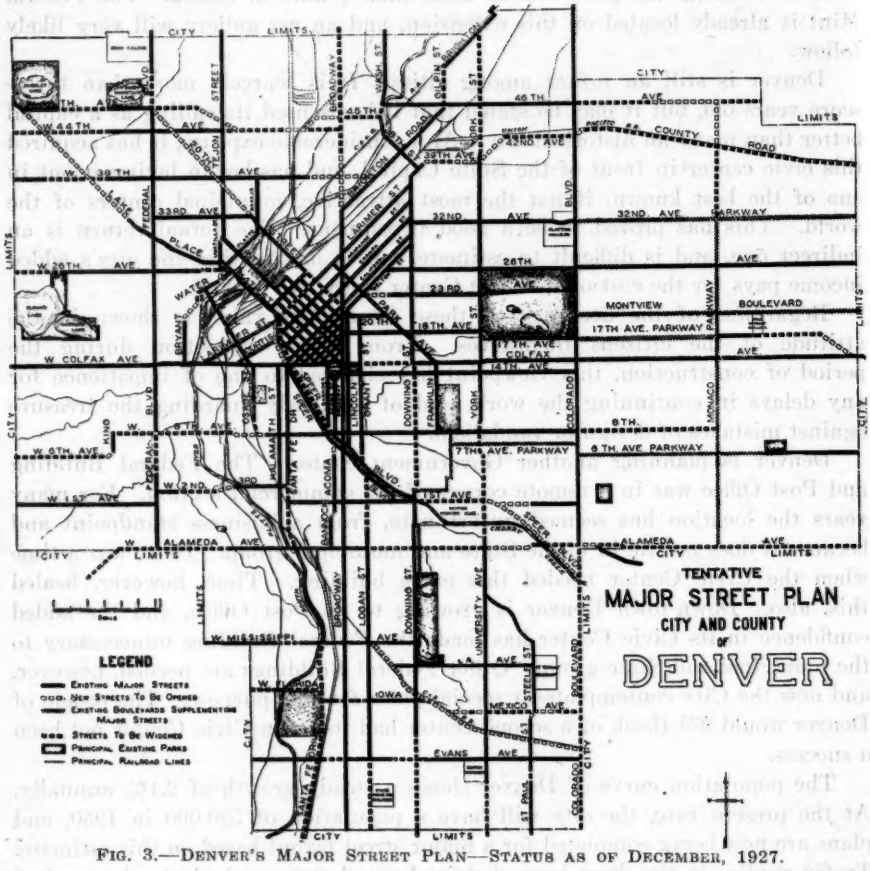

FIG. 3.—DENVER'S MAJOR STREET PLAN—STATUS AS OF DECEMBER, 1927.

Broadway is one of the major traffic streets of Denver. It was extended in 1919, and it is now (1929) contemplated to continue it to the city limits north. Many other radial lines are being considered, some of which will necessitate the building of viaducts over the Platte River. The main belt boulevard will be followed by other lines that reach out in large ellipses around the city. One of the boulevards follows the railroad lines, and ties together in the eastern part of the city on an existing parkway. This circular boulevard, in its down-town part, becomes a by-pass road for an east and west line.

Denver's recreational system includes approximately 1 664 acres of city parks and 11 006 acres of mountain park land. The City tries to maintain

its parks in the highest degree of beauty and efficiency. Numerous roadways have been eliminated from the larger parks, to save them from becoming short-cut routes. The traffic has been collected on arteries bordering the parks. The policy of using these border streets as major arteries of traffic has the advantage of creating unbroken traffic lines, and gives the motorist some benefit of the park without allowing him to destroy its quiet atmosphere.

Plans for a systematic playground development are under consideration. If these are adopted, the park and school playgrounds, combined, will furnish one playground for every ¼-mile radius. The school system of Denver is also being developed by means of carefully studied plans. A school is now provided for every ½-mile radius of the city.

In its zoning plan, Denver has adopted the principle of large areas for one-family homes. The city is built for light and air. Ground is cheap, and any one can afford to build on sufficient area. Height regulations existed in Denver before zoning ordinances were developed, with the result that the city has a uniform twelve-story skyline, harmonious with the idea of a capital city. The industrial district is located along the railroads, running in a narrow strip from the extreme southern limits of the city to the northern boundary.

Conclusions

In the development of the Denver Plan, there has been opportunity to determine just how far general planning principles, derived from the study of classic examples in European capitals, can be made to apply to American conditions. It is also interesting to determine, from the study of specific cases, what the broad fundamental principles of a problem seem to be, and to what extent they will be useful for general application. From experience at Denver and general studies made in conjunction therewith, it appears that the theoretical plan of a capital city can be divided into four major zones: The zone of Government buildings; the central business district; the residential zone; and industrial zones.

The Zone of Government Buildings.—This is an area not usually found on city zoning plans. The City of Washington, D. C., has this zone, divided into two focal points—the Houses of Congress in one, the Executive Buildings in the other. For a smaller State capital, it will not be possible to plan on such a large scale. The State Governmental group will usually consist of Capitol buildings, additional office buildings, Historical Museums, State Armory, and often a Federal Building. These buildings should be grouped around a large and attractive square. A second open square should contain all the buildings serving the municipality, including the City Hall, City Auditorium, Post Office, Library, Chamber of Commerce Building, etc. This second square should be also well designed and proportioned, but may be slightly less imposing than the former.

The Central Business District.—The two squares should be directly connected by one, two, or three broad shopping streets. In the planning of this shopping zone, the governmental character of the city must be kept in mind. The people of a Commonwealth want to be proud of their State House, of the grounds around it, and of the city surrounding all. Thus, the shopping zone must answer this requirement. Streets should be wide and building

heights should be well proportioned to the width. The building height limitation of the office buildings must be such as to avoid dwarfing the Capitol buildings.

The Residential Zone.—The residential districts should be built on similar lines. Here, more than anywhere else, must the capital city outshine every other city in the State. It must have attractive residential streets, ornamented by unusual horticultural treatment, with broad parkways, and curving roadways along creek and river beds. The apartment house zones should be placed around open squares of park. The residential district should be so inviting as to give every one of the State's citizens the desire to move to the capital city eventually.

Industrial Zones.—These should be carefully located away from the other zones, and should be on the outer border of the city. They can be made large enough to sustain a fairly large city. The residential areas essential to them should be placed between the industrial zone and the governmental area.

With the governmental square and the civic center located, the railroad station should be the third point of a triangle. The three features should be connected by direct and wide street lines and, possibly, by a boulevard running in a circle around the city's shopping center and connecting these three features. The station square must be attractive and set off by hotel buildings.

From each of these three center points, major traffic streets should run in all directions. Every point should have at least six different street lines radiating from it, and centering on the important building of its group—the State House, the City Hall, and the Railroad Station. The radiating street lines should be tied together by a circular boulevard surrounding the whole city.

This theoretical plan is naturally subject to the topography of the site. In case of the governmental group, it may be possible to select a high promontory, as was the case in Salt Lake City, where the State House towers over the Utah plains as the Acropolis once dominated Ancient Greece. Such an arrangement lends itself to great beauty, and a city should not fail to make use of it. The city center must bear a close relation to the shopping district, but there is no reason—if it were possible—why this square might not be on a second and lower hill, leaving the shopping center in between in a slightly lower area.

The time for low and relatively small governmental buildings will soon be passed. Capital cities must plan along bigger lines than they have dared to dream of thus far. The country has not reached its maximum growth, or population, by any means. At the rate of density of the population of present-day Germany, this country may eventually hold a population of 800 000 000 people. This means a sixfold increase, which is not only possible, but very probable. It will mean a proportionate growth in the cities, especially in the capital cities.

Denver is absolutely self-conscious in regard to its needs and its destiny. To be the capital city of a proud Commonwealth, it must be attractive for visitors; it must be good enough to live in; it must be clean and hospitable; and, even more, it must have the imposing character that the people can idolize.

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PAPERS AND DISCUSSIONS

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GOVERNMENT RECLAMATION POLICY IN BRITISH INDIA*

By E. S. Lindley,† M. Am. Soc. C. E.

This paper describes the British Indian land-holding and revenue system based on ancient custom; how, in the Punjab, water is given, and charged for; what land is reclaimed, and how. Without attempting a comparison with America, attention is drawn to points that seem to be of value. Data are given which show the magnitude of irrigation and reclamation works which seems not to be realized in America.

Efforts of the British Government in the Punjab cover nearly all the reclamation work of India. In a pamphlet, "Federal Reclamation, What It Should Include", Elwood Mead, M. Am. Soc. C. E., devotes a section to "What Other Countries are Doing". In this, facts and even mention of India and Egypt are conspicuously absent. This is not an omission that is unimportant, for it is believed that the 27 500 000 acres annually irrigated in India are more than one-half the irrigation of the whole world. Of that in India, more than 40% is in the Punjab; the 5 000 000 acres of desert Crown Waste populated by canals constitute more than 20% of the area served by existing canals in the Province, and schemes under execution or projected will double that area of reclamation. Moreover, these data do not include privately owned desert land. In what follows, costs have been translated into terms of American money, for convenience.

The Economic Problem and Its Solution

The reclamation policy of the Punjab Government has grown up from experience, starting from certain points of fiscal custom which must be described for proper understanding. Immemorial custom handed down from previous administrations prescribes that private "ownership" of land is subject to payment of the Government Land Revenue, or rent, as a first charge; in

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fact, in the last resort land may be sold to recover unpaid revenue. The owner may cultivate the land himself, or may let it to tenants. Such tenancies may be as firm as the ownership itself, subject only to payment of the agreed rent to the owner; or they may be ordinarily terminable at will.

Certain large desert tracts that were uncultivable because of their aridity were "Crown Waste", subject to no rights of ownership. Questions of "reclamation", called "colonization" in the Punjab, arise only where such Crown Waste is brought under irrigation.

The Punjab covering 65 000 000 acres, inclusive of mountains and areas with sufficient rainfall, 21 753 000 acres are within the irrigation limits of Government canals already completed. Of this, 4 726 000 acres were formerly Crown Waste; 11 160 000 acres was the average annual irrigated area (measured field by field) in 1922; and the tendency of growth is still upward apart from the opening of new canals. Canals now under construction in the Sutlej Valley Project provide for covering 8 878 000 acres, of which, 3 821 000 acres are Crown Waste; other canal projects which have been framed for successive execution will bring the area covered to a total of 35 327 000 acres, of which 9 674 000 acres in all are, or were, Crown Waste. The largest single block of Crown Waste was on the Lower Chenab Canal—2 300 000 acres out of a total of 3 391 000 acres covered; this canal irrigates more than 2 500 000 acres annually.

The customary revenue taken by pre-British Administrations was "in kind", by division on the threshing floor; the fraction taken varied, but was most commonly one-sixth of the gross produce, at least in theory. The British Administration soon based its demand on the net produce, and substituted cash rents; the fraction applied to this smaller amount was soon fixed in theory at one-half. The British claim may be described as half the economic rent in theory, but actually only from one-third to one-quarter is charged. The change to cash rents based on net produce obviously involved a system of assessment of agricultural profits. Such assessment, called a "settlement", is repeated at intervals, generally of twenty years.

Where the conditions affecting agriculture are relatively stable, the revenue to be levied is fixed and distributed over all the cultivable land; in bad years, its collection may be postponed, or even remitted. Where the conditions are less stable, the revenue is reduced to acreage rates to be levied season by season on the fields actually cropped; and in bad years the revenue may be reduced or remitted.

Proprietary lands to which irrigation is extended are generally under the latter ("fluctuating") form of assessment, because rainfall is uncertain. For land that can be given water, the "dry assessment" is then increased by a "water advantage rate", this being the revenue on the increase of rental value due to irrigation facilities. The total "wet" assessment for such land is now about $1.70 per acre in the year when cropped.

Water is also charged for at acreage rates per crop season, levied only when water is actually used; these average about $1.70, varying from $1 to $4 according to the crop grown. These rates are fixed partly on the value of the crop and partly on its water requirements. They are deliberately fixed very

much below the commercial value of the water, which must approximate what it would cost a cultivator to water his land from a well with bullocks, say, $9 per acre per crop. On a small group of private inundation canals, instead of the water rate, the proprietors take "chauharmi", one-quarter of the divisible produce on the threshing floor.

There are no water rights enforceable at law, on the supplies delivered by Government canals. It is the policy of the Government to supply water to all whose land is so situated as to be able to take it, to such extent as water is available. This policy is followed impartially. Occasional proposals to penalize villages or individuals by withholding water, have invariably been disallowed, and frequent discontent is caused by refusals to reward loyalty or services by extra water. A project now under execution provides largely for irrigation in States under Indian Princes. Apart from the duty incumbent on any civilized administration, its interests are to exploit resources to the greatest benefit of the community as a whole. The degree of these interests can be judged from the fact that the net revenue derived from the construction of canals, a return of nearly 20% on the capital invested, constitutes more than 40% of the total revenue of the Punjab.*

When irrigation is extended to a tract, it is divided into zones according to the depth from the surface to ground-water. For each zone a percentage of the total area is fixed for which water shall be provided; it is lower for proprietary land (which can be cultivated to some extent without canals) than for Crown Waste to be reclaimed (which it has not been possible to populate at all without a canal). These percentages range from 25 for proprietary land with ground-water within 20 ft. of the surface, to 75 for Crown Waste with ground-water more than 40 ft. from the surface. The ratio of irrigation proposed in the summer and winter seasons is also fixed, as well as the summer and winter discharges at heads of farmers' ditches, per acre of proposed irrigation. Higher rates of discharges are given to some poor lands which need more water. These quantities, applied to the area to be served by each farmer's ditch, give the discharge to be delivered there; the outlets at the heads of farmers' ditches are designed to deliver these discharges. If the summer is higher than the winter rate, the outlet is designed for that, and in winter the time of delivery is reduced according to the supplies actually available. That the designed rates of supply are liberal is shown by the irrigation accomplished; this is 1.21 times the proposed irrigation for the entire system of canals in the Punjab for which supplies are secured by weirs (85% of the whole).

Before the recent introduction of "semi-modules" for outlets, it was impossible to determine the average discharge—the criterion in judging complaints of insufficient supply was then the area actually irrigated. If an outlet were found to be irrigating less ground than proposed, and this was not accounted for by waste or by intentional limitation to support the complaint, the size of

* The Lower Chenab Canal, which brings in two-fifths of this revenue, pays more than 50%; but this is not profit calculated in a commercial way. Water rates alone minus working expenses are 10% for the whole Province and 22.5% for the Lower Chenab Canal. The remainder is accounted for partly by enhancement of land revenue for irrigation facilities, and partly by land sales; it is not clear whether the last is interest on total realizations to date, or instalments received during the year.

the outlet would be increased (after approval by competent authority, and not merely by the local assistant engineer). Some engineers would even admit as the criterion not merely the proposed irrigation, but the ratio actually achieved in the neighborhood. With the introduction of "semi-modules" the criterion is becoming the discharge actually delivered, as it should be; this has the advantage of simplicity, and of no longer penalizing care and efficiency in irrigation.

When a new canal is opened, it is at first possible and necessary to give more than the designed supply to the land; this is gradually reduced as more land is developed. Under the conditions described, the originally designed area does not form the limit of development that is possible with a given supply of water. In some areas the ground-water level rises sufficiently to limit the supply that can be applied without harm, or even to water-log the land. In some cases actual irrigation has risen so far above what was proposed, as the result of improved distribution by the Government engineers (and, probably to a lesser degree, better irrigation by the farmers), that supplies can also be reduced. There is nothing to limit such reductions of present supplies, beyond the need to give fair and equitable treatment to present irrigators.

Experience with the Lower Chenab Canal

The Lower Chenab Canal Colony is a good example of Punjab reclamation. It had 2 250 000 acres of Crown Waste, and serious inaccessability. The small blocks that were colonized previously in other places had afforded some valuable experience, although reclamation problems can hardly be said to have arisen. Since then, other areas have been colonized, some of them considerable, but principally after the Lower Chenab Canal had become an assured success; these will be referred to in noting points of difference.

The Province was predominantly agricultural, about 90% of the population being occupied with farming, and the inhabited parts were sufficiently crowded to cause land hunger. Peasants, however, are conservative—where the margin between earnings and needs is narrow and precarious, an Oriental feels less the impulse to rely on his powers and try to better himself, than the fear of losing even what he has. "A bird in the hand is worth two in the bush". Canals had not then achieved the conspicuous success they now have, and a serious failure might have stranded the whole population nearly a week's march even from drinking water. However, the Administration did not need to go beyond its own borders to find settlers.

The total of settlers required was divided into quotas for each district, for which the Deputy Commissioners in charge of the Civil Administration were asked to nominate candidates. These quotas were presumably determined by relative degrees of crowding, and the quality of the peasanty. In some districts more difficulty was experienced in finding settlers than in others; in fact, it is often claimed that some districts made this an opportunity to export some of their undesirable characters; thus, the original quotas were not rigidly exacted.

In previous schemes settlers had also not been taken from the adjacent surroundings; in one case because of local opposition to the scheme; in the other because it was found that the attempt injured the proprietors by drawing off their tenants. In later schemes, it has gradually become a matter of selection from the crowd of eager applicants.

Settlers were sought for land grants of three orders of size: Peasants, to receive from one-half to two unit "squares" of 28 acres each; yeomen, to receive from three to five squares, which they would cultivate themselves with the assistance of hired labor; and capitalists, who were to receive larger grants to lease to tenants, and were intended to become the gentry and natural leaders of the new community. A small proportion of the area was sold by auction. In the first blocks, 67.5% of the area was granted to peasant colonists, 15% to yeomen, and 10% to capitalists. In later blocks, the first class was increased at the expense of the two latter, as these were found to be less satisfactory colonists.

The terms of course included payment of water rates, then about 80% of the present rates (but on a new canal water is always given free for the first few harvests); also land revenue, although this seems to have been only about $0.50 for the first years. In addition to this revenue, corresponding to the revenue in proprietary lands, there was a payment to the Government as the proprietor. Yeomen had to pay $2 per acre in two instalments, and capitalists from $3.30 to $6.60 on entrance; and both had the option to acquire proprietary rights after 5 years for $6.90 per acre. Peasants were given tenancies for 20 years, with the option of acquiring permanent tenancy rights within 5 years and without payment; they were charged an annual royalty, apparently only $0.125 at the start, in lieu of the rent they would have had to pay to a proprietor. About 20 years afterward, these peasants were given the option of acquiring proprietary rights for $4.20 per acre, payable in instalments.

In addition, peasant grantees were from the beginning required to reside on their grants. Very soon the same condition was laid also on yeomen grantees, and the building of a house was further explicitly enjoined. All were required to join in digging a village well, in keeping the village less unsanitary than is customary in the country, in protecting the common land (roads, farmers' ditches, grazing grounds, etc.) from encroachment, and in planting a few trees. To protect them from their own improvidence, rights of alienation were restricted; and to keep control of the type of settler the customary rules of inheritance were limited.

In effect, the settlers were given the bare ground to start on. Boundaries had been demarcated, a village site and roads were located, and canals and farmers' ditches had been made ready to bring water to a point in each holding. All else, from clearance of brush to the building of houses, was left to them.

In addition to the Deputy Commissioner appointed to ordinary civil charge of the new area, and the Canal Staff for technical duties and canal administration, a Colonization Officer of the Indian Civil Service was appointed for work in connection with selecting the settlers, allotting them

their grants, seeing that they fulfilled their conditions, and all the work incidental to establishing the new population.

The writer has not ascertained the prices at which land was sold in the early colonization of this area. In 1896, when this colony was just beginning a rapid increase of irrigation, an average price of $17 per acre was realized in another small and fairly accessible colony; while in 1904 the average price of a further small auction in the same colony was $89 per acre. Now (1928), the average rent commanded by land on the Lower Chenab Colony is about $6.60 per acre, while good land favorably situated commands three times as much. About 1916 land in the Lower Bari Doab Colony was sold during a land boom, and on occasion more than $200 per acre was paid, for new unbroken land. The fee for which peasants were allowed to acquire proprietory rights had gradually been raised, and was then $33.30.

To a proportion of grants on the Lower Chenab Canal special conditions were attached; some grantees were required to maintain a small area planted in trees; some were required to maintain a suitable mare for breeding remount horses or transport mules, and to give the Army the first right of purchase. Many of the "Janglis", the former nomad grazing tribes who were the only inhabitants of the desert, were required to continue to maintain camels, and supply transport for Army use when needed. In the Lower Jhelum Colony the bulk of the grants were made on horse-breeding conditions. An effect of this has been that the settlers are, on the average, a less satisfactory type of farmer.

In the earlier schemes, a proportion of the grants was earmarked for ex-soldiers and the races with traditions of military service. The colonization of the Lower Bari Doab Canal was still proceeding when the main recruiting effort for the European War was made, and to assist this the area earmarked for military grants was considerably increased. Including the scheme now under execution, a total of 387 000 acres has thus been specially set aside or given in military grants. This area does not include the grants mentioned previously, or special good service rewards of land given free of revenue demands.

In the earlier schemes only small areas were auctioned. As colonization of the Lower Bari Doab Canal was still proceeding when the period of difficult war and early post-war, finance was entered, land sales were increased to meet the difficulty. In the scheme now under execution, land sales are providing an appreciable proportion of the capital needed for construction, thus decreasing the amount that has to be raised in loans.

The experience and now assured success of the older colonies make a modern colonization simple when compared with earlier efforts. Some of the more obvious difficulties can easily be imagined. Before the colonist on the Lower Chenab Canal had settled down at all, a severe epidemic of cholera swept over them, and the Lower Jhelum colonization was interrupted by plague. If allowance is made for picturesque description, some of Kipling's stories convey an idea of such situations, with a handful of Englishmen and their womenfolk heading a staff of Indians to manage a scared crowd. The

Janglis did their best to wreck the scheme; they claimed ownership of the tracts they had roamed and grazed over, and the earlier administrators too lightly ignored the question of what to do with them, although the nomads of other deserts naturally would not admit them. One of their standing occupations was cattle theft, and by this and in many other ways they persecuted the settlers. This drove many away, and others frankly declared their intention of leaving as soon as free water ceased. However, the scales were turned when some years of good rainfall up to 1894 were succeeded by lean years from which the canal protected the tract; relatives and labor flocked in, and the settlers began to make good profits. Soon afterward a railway into the colony was opened and this improved the marketing of produce.

An internal question in this work of colonization and canal administration has been the fairness of the terms charged. The first colonization was so experimental that it was necessary to offer extremely liberal terms at the start, and to think more of the point of view of the settler than that of the taxpayer. As success was achieved, it became possible to take a wider view, and terms have gradually been raised. The question is debated from two points of view: the settlers party assert that the high rate of profit from these works means that they are being called upon to bear an undue burden: the opposite point of view is that these profits are really a measure of the extraordinarily favorable conditions for canal development, and that the charges are only a fraction of the true value of what is given. The settlers are drawn from the agricultural population of the whole Province, which is the bulk of the whole population; and, much of the Province being under canal irrigation, the bulk of the agriculturists are equally interested in low water rates. Opposition should come from those who are not in a position to enjoy cheap water and colony land—those whose taxation would be lightened if the charges were raised. Actually, this opposition is hardly vocal, and the Government has at the same time to voice the opposition and to judge the issue fairly. As a recently retired official of that Government, the writer simply presents these statistics without personal comment.

Lessons Learned from Indian Reclamation

Without attempting comparisons with American conditions, it seems worth while to draw attention to some points that appear of value.

India has the obvious advantage, in its agricultural population, of a reservoir of settlers of the right stamp. Success has been greatly facilitated by the liberal terms given them; but these were made possible only by favorable natural conditions—ample supplies of water which could be brought cheaply to large blocks of flat fertile country.

By comparison with provinces in which canal engineers were restricted to the technical maintenance of channels, it has been found advantageous to make the engineers responsible also for revenue work, as that brings them into intimate touch with the results of their technical labors.

Instead of the canal management being in the hands of the farmers, who need consider only their own interests, it is in the hands of the Administration which is impartial. The branch of the Administration immediately concerned

is often accused of bias, of considering too much the showing of profits to mark its efficiency; but it has not the sole voice in the Administration, and the data quoted make it possible to judge whether there is bias shown. Impartial management is an advantage toward securing efficiency.

Given an impartial administration, it is an advantage to have water rights in such a form that fair development is not checked. The system of water rights described is advantageous in permitting the exploiting of supplies greatly in excess of the minima.

It is disadvantageous that irrigation receipts go straight into general revenue, and that funds for improvements depend on the general budget situation. This disadvantage in the case of Indian State Railways has recently been admitted and removed.

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ONE HUNDRED FIFTY YEARS ADVANCE IN STRUCTURAL ANALYSIS

Discussion*

By H. M. Westergaard, M. Am. Soc. C. E.†

H. M. Westergaard,‡ M. Am. Soc. C. E. (by letter).§—The writer has read with interest and appreciation the discussions by Professor Timoshenko,‖ and Messrs. Flinn¶ and Feld.** Indebtedness is expressed to Professor Timoshenko for his account of the Russian tradition.

The writer does not understand the remark by Mr. Feld about Cauchy and Lamé. His reference to an investigation by S. E. Slocum, M. Am. Soc. C. E., published in 1911, to determine the influence of the shearing stresses in beams upon the deflections, by consideration of the energy, may be supplemented by the statement that W. Ritter solved the problem in 1888 by a simpler consideration of the energy, and that R. Land presented a particularly satisfactory simple solution of the whole problem in 1894.††

Professor Timoshenko referred to the work of Boobnoff in the theory of the structure of ships, and Mr. Feld mentioned the structural problems in aeronautical design. It is significant that the applications of structural mechanics extend to many fields of engineering. In aeronautical design the emphasis on economy of weight has led to new problems, involving, especially, the possibility of failure by buckling.‡‡

* Discussion of the paper by H. M. Westergaard, M. Am. Soc. C. E., continued from October, 1928, *Proceedings*.

† Author's closure.

‡ Prof. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

§ Received by the Secretary, April 1, 1929.

‖ *Proceedings*, Am. Soc. C. E., October, 1928, Papers and Discussions, p. 2380.

¶ *Loc. cit.*, p. 2384.

** *Loc. cit.*, p. 2385.

†† W. Ritter, "Anwendungen der graphischen Statik," v. 1, 1888, p. 148; R. Land, "Einfluss der Schubkräfte auf die Biegung statisch bestimmter und die Berechnung statisch unbestimmter gerader vollwandiger Träger," *Zeitschrift für Bauwesen*, v. 44, 1894, Columns 611–626.

‡‡ For example, on the subject of wing-beams, see, H. Reissner and E. Schwerin, "Die Festigkeitsberechnung der Flugzeugholme," Jahrbuch der Wissenschaftlichen Gesellschaft für Luftfahrt, v. 4, 1916; H. Müller-Breslau, "Graphische Statik der Baukonstruktionen," v. 2, sub-vol. 2, Second Edition, 1925, pp. 647 and 661; and Joseph S. Newell, Jun. Am. Soc. C. E., "The Design of Airplane Wing-Beams," *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 725.

Professor H. Reissner, of Berlin, Germany, in a letter to the writer, expressed the view that certain chapters in the history of structural analysis might well have been included in the paper, or dealt with more fully. He mentioned the following subjects: Vibrations,* buckling,† airplanes, and experimental devices for structural analysis. The writer agrees with Professor Reissner, but wishes to add that many more subjects might have been included.

Referring to the experimental devices, the introduction of the use of polarized light by A. Mesnager, for the purpose of determining stresses, was mentioned in the paper, but it should have been stated that E. G. Coker has done eminent, extensive work with this method‡ The use of models with the devices invented by George E. Beggs, M. Am. Soc. C. E., was mentioned in the paper. The soap film analogy according to L. Prandtl, for the study of torsion of non-circular members, was mentioned. This analogy has been supplemented by a roof-and-membrane analogy indicated by A. Nádai, whereby one may study, experimentally if one wishes, the progress of plastic action or yielding that must take place in a non-circular member subject to torsion.§ Lydik S. Jacobsen has devised a simple and effective method of determining the stresses produced in a circular shaft with variable diameter by measuring electric potentials in a plate of non-uniform thickness.‖

The subject of secondary stresses in trusses might well have been discussed in the paper. At this place, it will be sufficient, however, to refer to the recent notable paper on the subject by Cecil Vivian von Abo, Assoc. M. Am. Soc. C. E., which contains an historical discussion.¶

An important principle, which was not mentioned in the paper, was stated by Melan in 1888, and applies to suspension bridges.** A heavy chain possesses flexural resistance by virtue of its weight. Applying this structural principle to the bridge, one finds that the stiffening trusses are aided by the dead load.

In concluding, the writer wishes to call attention to a recent development in the theory of beams. In order to deflect vertically, without twisting, a channel used as a beam with the web vertical should be loaded so that the

* It is proper to mention in this connection the following paper by H. Reissner: "Schwingungsaufgaben aus der Theorie des Fachwerks," *Zeitschrift für Bauwesen*, v. 53, 1903, Columns 135–162. On the general subject of vibrations, see, W. Hort, "Technische Schwingungslehre," 1922 (bibliography on vibrations of trusses, p. 802), and S. Timoshenko, "Vibration Problems in Engineering," 1928.

† A bibliography on the subject of buckling may be found in *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 650. The following more recent investigation should be mentioned: R. v. Mises, "Ueber die Stabilitätsprobleme der Elastizitätstheorie," *Zeitschrift für angewandte Mathematik und Mechanik*, v. 3, 1923, p. 406. The same volume contains four other notable papers on buckling.

‡ See, for example, E. G. Coker, "Some Engineering Problems of Stress Distribution," *Proceedings*, First International Congress for Applied Mechanics, Delft, 1924 (pub. 1925), p. 18.

§ A. Nádai, "Der bildsame Zustand der Werkstoffe," 1927, p. 97.

‖ Lydik S. Jacobsen, "Torsional-Stress Concentrations in Shafts of Circular Cross-Section and Variable Diameter," *Transactions*, Am. Soc. Mech. Engrs., v. 47 (1925), p. 619.

¶ Cecil Vivian von Abo, "Secondary Stresses in Bridges," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1. See, also, Z. Bazant, "Influence du système de triangulation sur les efforts secondaires," Académie Masaryk du Travail, *Publication Scientifique No. 15a*, Prague, 1923, and "Les efforts secondaires des poutres en treillis," *Loc. cit.*, *Publication Scientifique No. 37a*, Prague, 1926.

** Leon S. Moisseiff, "The Towers, Cables and Stiffening Trusses of the Bridge Over the Delaware River between Philadelphia and Camden," *Journal*, Franklin Inst., October, 1925.

resultants of the loads lie in a certain vertical plane. This plane does not contain the centers of gravity of the cross-sections, nor the centers of the web, but is found outside the beam behind the web. The strikingly simple explanation was found by A. Eggenschwyler* and R. Maillart.† In the cross-section one finds vertical shearing stresses in the web and horizontal shearing stresses in the flanges. The resultant of the horizontal shearing stresses is a couple, which, combined with the resultant of the vertical shearing stresses, gives as the resultant of all the shearing stresses a vertical force lying in a plane outside the beam. The resultants of the loads, obviously, must be contained in this plane, if the beam is to deflect without twisting. Tests‡ with steel channels have shown this conclusion to be accurate, but one may also perform the experiment convincingly with a model made of cardboard. It is strange that a structural principle so simple and important, and so easy to demonstrate, should have remained unnoticed until only a few years ago.

* *Schweizerische Bauzeitung*, v. 76, 1920, pp. 206 and 266; *Eisenbau*, 1921, p. 207; *Zentralblatt der Bauverwaltung*, v. 41, 1921, p. 501; *Der Bauingenieur*, v. 3, 1922, p. 11.

† *Schweizerische Bauzeitung*, v. 77, 1921, p. 195; v. 83, 1924, p. 109; see, also, A. Ostenfeld, "Teknisk Elasticitetslære," Fourth Edition, 1924, p. 404; A. and L. Föppl, "Drang und Zwang," v. 2, Second Edition, 1928, p. 122.

‡ Especially by Bach; see the papers by Eggenschwyler and Maillart referred to previously.

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TRANS-MOUNTAIN WATER DIVERSIONS

A SYMPOSIUM

Discussion*

By H. A. Van Norman, M. Am. Soc. C. E.†

H. A. Van Norman,‡ M. Am. Soc. C. E. (by letter).§—The writer wishes to thank the discussors for the interest which they have shown in the proposed Los Angeles-Colorado River Aqueduct. Many of the ideas suggested are feasible under some one or more variations of the conditions of the problem which are as yet not specifically defined.

However, several additional steps have been made toward the realization of this gigantic and needed improvement, two of which are worthy of mention. The first step occurred on November 6, 1928, when eleven incorporated cities, in the Southern California Coastal Plain, signified their intention, by popular vote, of joining a Metropolitan Water District of Southern California for the purpose of procuring water from the Colorado River. The second great stride was the passing of the Swing-Johnson Bill on December 21, 1928. The significance of this Bill is that the Federal Government definitely decided its policy in the allocation of water from the Colorado River and made the first step toward development by the authorization of the construction of the Boulder Dam. The location of the first dam to be built on the river necessarily fixes a definite outline for future development. This eliminates one of the heretofore variable conditions in the plan of any Los Angeles-Colorado River Aqueduct.

The construction of a high dam at Black Canyon by the Federal Government, which will completely regulate the flow of the Colorado River and prevent the major portion of the silt from reaching any smaller reservoirs which may be built below it, allows the alternatives not mentioned in the paper,‖ to be considered.

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* Discussion on the Symposium on Trans-Mountain Water Diversions, continued from November, 1928, *Proceedings*.

† Author's closure.

‡ Gen. Mgr. and Chf. Engr., Dept. of Water and Power, Los Angeles, Calif.

§ Received by the Secretary, May 15, 1929.

‖ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1116.

Blythe-Parker Route.—Under this condition, a dam approximately 80 ft. in height might be built at one of two favorable locations above Parker, Ariz. The first site is situated just below the junction of the Williams River and is about 12 miles above Parker. The other site is about 6 miles up stream from Parker. An 80-ft. dam at either place would create a reservoir of approximately 1 000 000 acre-ft., which would raise the water surface to Elevation 440 and re-desilt the water for the aqueduct. From a reservoir so created, a line could be built along the west side of the river to connect with the present Blythe route south of the infiltration gallery. An alternative would come down the river front to the north end of the Riverside Mountains where a pump lift would raise the water to Elevation 940. Thence, it would proceed westerly and southerly, *via* Grommet and Rice, Calif., tunneling through the Coxcomb Mountains, to a junction with the Blythe route, at an elevation of 1 300 ft., about 12 miles east of Shaver's Summit.

Pichaco Route.—This proposed route originates in a reservoir about 30 miles up stream from Yuma, Ariz. The reservoir would be created by a dam of undetermined height to be built in a canyon of the Colorado River, about 3 miles below the old mining town of Picacho, Ariz., where the water surface has an elevation of 170 ft. It would have a capacity of from 1 000 000 to 2 000 000 acre-ft., depending on the height of the dam. Drillings are now (1929) being made to determine to what height it is feasible to construct a dam at this point. Enough power can be generated at the dam to lift the water to Elevation 500.

From the upper end of a pump lift at Elevation 500, the aqueduct pierces the Picacho Mountains in a tunnel approximately 14 miles long, with shallow shafts, to the east slope of the Salton Sea Basin. Thence, it continues northwesterly in conduit through smooth desert country along the eastern slope of the basin to a point east of Mecca, Calif., where there is a 10-mile stretch of short tunnels. Thence, the line proceeds in conduit to a point 6 miles northeast of Indio, Calif., where the water is lifted approximately 1 025 ft. to Elevation 1 250, continuing in conduit section to 1 mile west of Whitewater, Calif., where there is another pump lift of 275 ft. to an elevation of about 1 500, at which point it joins the Blythe route or a variation thereof. From this point the aqueduct follows the Blythe route under San Gorgonio Pass and to the Metropolitan Water District.

The Picacho route, if constructed, would be about 242 miles long from the Colorado River to a terminal reservoir at or near San Gabriel, Calif., and would require a total lift of approximately 1 580 ft. This route throughout its length would be through accessible country, in the proximity of the Southern Pacific Railroad except at the point of diversion.

The statement by Mr. Jakobsen* as to the ultimate life of the reservoir at Bridge Canyon is correct if one assumes 100 000 acre-ft. of silt per annum, and that the full volume is effective as a desilting medium; but if the figure of 137 000 acre-ft. of silt per annum, as given by Fortier and Blaney in their more recent studies, is used, and only a part of the volume of the reservoir is considered for silt storage, then the effective life of a reservoir at Bridge Canyon would be greatly reduced.

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2582.

Mr. Jakobsen also states* that by capitalizing the annual pumping cost which would have to be met on any of the pumping routes, at 4½% per year, the money value would be $187 000 000. This would be nearly cut in half if he would use an interest rate of about 7 to 8%, in order to offset depreciation, operation, and maintenance charges on the excess length and cost of the gravity aqueduct with which he is making his comparison. It would be further reduced if consideration were given to the fact that during the first years of operation the annual pumping costs are less than one-half those under full development of the project.

Mr. Bennett† suggests a more southerly diversion from the Colorado River and the joint use of the Imperial Canals. Such a route would have the lowest initial cost of any of those described for the proposed aqueduct; but with the great amount of pumping and the necessity of desilting the water by settling basins or other means, it would be the most expensive to operate and maintain.

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2582.
† *Loc. cit.*, September, 1928, Papers and Discussions, p. 2196.

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ENGINEERING FOUNDATION COMMITTEE ON ARCH DAM INVESTIGATION

ARCH DAM INVESTIGATION

Discussion*

By Messrs. Roy W. Carlson, and J. L. Savage and Ivan E. Houk.

Roy W. Carlson,† Jun. Am. Soc. C. E. (by letter).‡—Since the report was published, Fred A. Noetzli, M. Am. Soc. C. E., has suggested that the method of taking successive differences of deflections to obtain moments, shears, and loads in cantilever sections as used by W. A. Slater, M. Am. Soc. C. E., in Table 24,§ might be adapted for designing an arch. The principal advantage sought is that of starting from an assumed deflection curve rather than from an assumed load distribution.

The writer has attempted to use this method in studying the stresses in the Pacoima Dam (constant-angle arch type). A crown deflection curve was assumed and the load corresponding to the deflection at each elevation was computed: (1) For the arch, by an accepted formula; and (2) for the cantilever, by the method of successive differences of deflections. It required patience and judgment to choose a deflection curve that would yield a load distribution within the bounds of reason, because four differentiations (or successive differences) were required, each introducing greater irregularities.

After exhausting the possibilities of taking differences, the process was reversed, making it a method of cumulative sums rather than successive differences, and working from load to deflection, rather than from deflection to load. As before, the first step was to assume a deflection curve, but arch loads were computed from this curve and subtracted from the water load to obtain cantilever loads for the "cumulative sum" process. The principal

* Discussion on the Report of the Committee of Engineering Foundation on Arch Dam Investigation, continued from May, 1929, *Proceedings*.

† Testing Engr., Los Angeles County Flood Control Dist., Alhambra, Calif.

‡ Received by the Secretary, April 22, 1929.

§ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 178.

advantage of the method was retained thereby. Table 52, which has been arranged by the writer, illustrates the method.

The elevation of each section above the base is given in Column (1); Column (2) gives the assumed arch deflection; in Column (3) is a tabulation of arch loads, computed by some good method from the arch deflections; and in Column (4) is listed the difference between the arch load and full water load, or the load left for the cantilever.

TABLE 52.—Analysis of Stevenson Creek Dam by Method of Cumulative Summation of Loads.

Elevation.	Assumed arch deflection, in feet.	Arch load, in pounds per square foot.	Cantilever load, in pounds per square foot.	$\frac{\text{Cantilever shear}}{5}$ in pounds.	$\frac{\text{Cantilever moment}}{25}$ in foot-pounds.	$\frac{625}{EI}$	$\frac{25 \times 10^5 \times \text{moment}}{EI}$	$5 \times 10^5 \times$ slope.	Deflection, in feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
60	0.0113	190	—190	0	0	0.00 81	0	.....	0.00730
				— 90				—475	
55	.....	.....	—160		— 90	0.00181	— 16		0.01205
				—250				—459	
50	0.0162	760	—140		— 340	0.00181	— 62		0.01664
				—390				—397	
45	.....	.....	— 30		— 730	0.00181	—132		0.02061
				—420				—265	
40	0.0211	1 180	70		—1 150	0.00181	—208		0.02326
				—350				— 57	
35	.....	.....	380		—1 500	0.00181	—272		0.02383
				30				229	
30	0.0205	1 190	680		—1 470	0.00181	—266		0.02154
				710				495	
25	.....	.....	1 100		— 760	0.00132	—100		0.01659
				1 810				595	
20	0.0105	980	1 520		1 050	0.00083	87		0.01064
				3 330				508	
15	.....	.....	1 580		4 380	0.00040	175		0.00556
				4 910				333	
10	0.0022	1 470	1 650		9 290	0.00017	158		0.00223
				6 560				175	
5	.....	.....	2 700		15 850	0.000080	127		0.00048
				9 260				48	
0	0	0	.....	.....	25 110	0.000034	85	.....	0

Column (5) contains a summation of the values in Column (4). The values given represent one-fifth of the shear at each foot of elevation. This is due to the fact that the shear is computed by the summation of loads at 5-ft. intervals instead of 1-ft. intervals. In a high dam, 20-ft. intervals are sufficiently accurate, and in such a case the values found by summation would be equal to one-twentieth of the actual shear at each 20-ft. interval. In beginning the summation, it should be remembered that the shear at the crest must be zero and at the center of the first 5-ft. interval it is two and one-half times the average load for the first 2½ ft.

Column (6) is a tabulation of bending moments and is a summation of the values in Column (5) from the crest down. The one-fifth factor enters again. Column (7) gives constants for changing bending moments into

pure bending units. The factor, 625 (5 raised to the fourth power), is used, so that no more multiplications are necessary in order to obtain deflections.

In Column (8) is listed the bending units which are the products of values in Columns (6) and (7), the decimal point being translated five places to the right so that whole numbers can be used. Column (9) tabulates the slopes which are obtained by successively adding values from Column (8), beginning at the base where the slope is known to be zero for a cantilever fixed at the base.

The deflections in Column (10) are obtained by beginning at the base and adding successively the slopes given in Column (9). The decimal point is restored to its proper place so that these values may be compared with the assumed arch deflections in Column (1). If they do not check sufficiently well, assumed deflections can be adjusted and cantilever deflections recomputed until agreement is obtained. Although the deflections shown in Table 52 do not check very well at the 60-ft. elevation, a change of 10 lb. per sq. ft. in the load near the crest makes poorer agreement, so they are sufficiently close.

Stresses in the cantilever can be determined from the moments in Column (6) by the well-known relation between bending moment, thickness, and stress. A slight correction for taper is advisable near the base. The stresses in the horizontal direction are computed by any reliable set of arch equations, and by applying to each arch ring its share of the water load as given in Column (3).

Since the writer wished to investigate the behavior of the Test Dam under a variety of assumptions, no established arch theory was used, but simple equations of sufficient exactness were developed as desired. Each arch ring was considered as a straight beam as far as bending loads were concerned, and the shears, bending moments, slopes, and deflections due to a unit uniform load were obtained by successively integrating the load and evaluating constants. Thus, the slope at the abutments could be arbitrarily chosen, and this was deemed important for checking the behavior of the Test Dam, since the abutment slopes were not zero (partly due to the fact that the concrete was cracked away from the abutments before loading).

Having obtained the deflection curve for each arch ring by integrating a unit uniform bending load, the rib-shortening of the arch, thus deflected, was computed from the average deflection in order to obtain the direct thrust and the load carried by that arch. The latter load was added to the unit bending load and the sum was designated to be the total load which caused the deflection curve previously mentioned. In computing the rib-shortening, a correction could be made for the yielding of abutments and closing of cracks, thus duplicating other conditions which prevailed at the Test Dam.

Thus far, the effect of shear on the deflection has been omitted. Since the solution of shears is included in the solution of cantilever deflections, it is a simple matter to calculate the deflection due to shear and add it to the bending deflection. For the thin Stevenson Creek Dam, an analysis including shear was made and so little difference was found that it was decided to omit that refinement from this discussion. The main effect of shear was to

increase the deflections in the lower part of the dam, thus throwing greater load on the lower arches.

This method of dividing load between arches and cantilevers is adequate for determining stresses in an arch of such design that the load on each arch ring is uniform from crown to abutment. This condition rarely exists in a service dam, although in many cases the load variation is slight. As an example, the Sawpit Arch Dam, located in Los Angeles County, California, gave cantilever deflections practically equal to arch deflections at all parts of the dam, with uniform load on each arch ring.

If a cantilever some distance from the crown is investigated, using the same loads as are found to give equilibrium at the crown, in general, it will be observed that the deflections do not agree with the arch deflections. The problem is to determine the variable horizontal loads which give deflection agreement for all parts of the dam.

The one-cantilever method herein described has been extended to permit a solution of this problem. In general, an arch is treated as a cantilever, except that load is carried by direct thrust as well as by bending. In the special case of a symmetrical arch, such as the Test Dam, it is simpler to express the load on an arch ring as a combination of a uniform load and a load varying according to some simple mathematical equation. The latter can be chosen so that it produces bending without direct thrust. This is accomplished mathematically by equating the integral of the deflection curve to zero.

In his studies of the Stevenson Creek Dam, the writer found that a variable load on the arch rings at the 50-ft. and 60-ft. heights and for uniform load below the 50-ft. level gave a satisfactory solution. A slight load varying as the three-halves power of the distance from the crown was added to a uniform load after it was found that the addition of a second degree variation gave too sharp a change toward the abutment. This equation was integrated five times; the first giving shear; the second, moment; the third, slope; the fourth, deflection; and the fifth, rib-shortening. A division by moment of inertia and modulus of elasticity was made to obtain slope, deflection, and rib-shortening in customary units. Conditions of slope at the abutment and the crown, deflection at the abutment, and shear at the crown were imposed to evaluate constants of integration.

Summary of Studies.—Four analyses were made. The principal ones involved the following assumptions:

1.—Arches and cantilevers were partly fixed at the abutments and the base, using slopes at the abutments and at the base as measured at Stevenson Creek. The closing of cracks and yielding of abutments as measured at Stevenson Creek were accepted as criteria. No reduction in area was assumed for the bases of cantilevers.

2.—Arches were fixed at the abutments and cantilevers were fixed at the base. There was no yielding of abutments nor closing of cracks. Conditions were purposely made to fit the celluloid model.

The first analysis gave deflections very much like those measured at Stevenson Creek at all elevations except within 10 ft. of the crest, where

the computed deflections were considerably smaller. Strains were in good agreement except around the two vertical cracks at the crown.

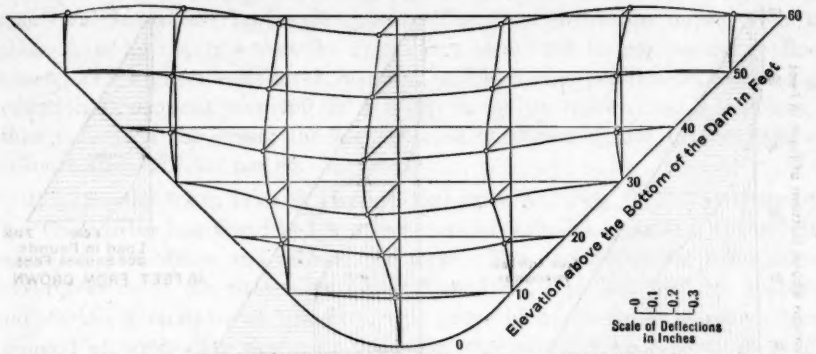

FIG 169.—ISOMETRIC DRAWING OF ANALYSIS DEFLECTIONS FOR 60-FOOT HEAD OF WATER.

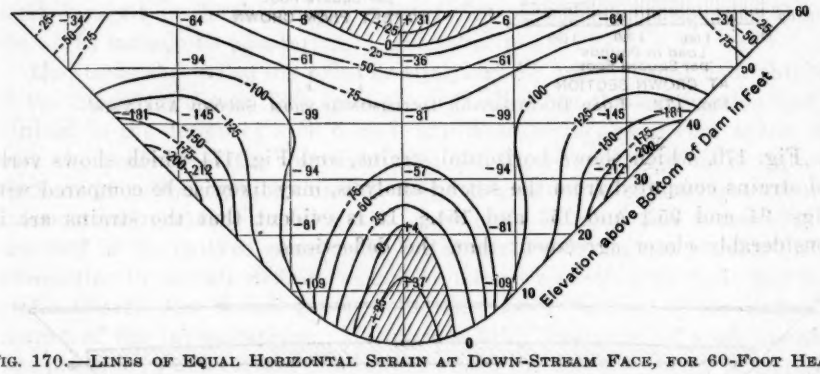

FIG. 170.—LINES OF EQUAL HORIZONTAL STRAIN AT DOWN-STREAM FACE, FOR 60-FOOT HEAD.

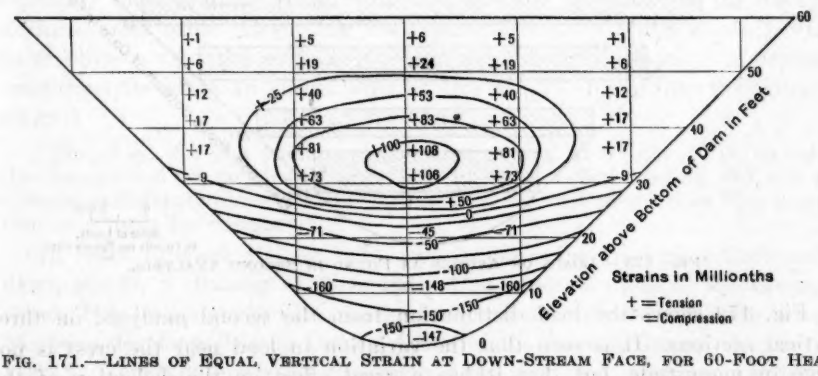

FIG. 171.—LINES OF EQUAL VERTICAL STRAIN AT DOWN-STREAM FACE, FOR 60-FOOT HEAD.

The second analysis gave deflection curves comparing better with those computed from the celluloid model than with the Stevenson Creek Dam, but with two noticeable differences. The deflections at all points were less than those shown by the model and at the crest this condition was exaggerated.

Fig. 169 which shows the deflections from this analysis, should be compared with Figs. 86* and 141.†

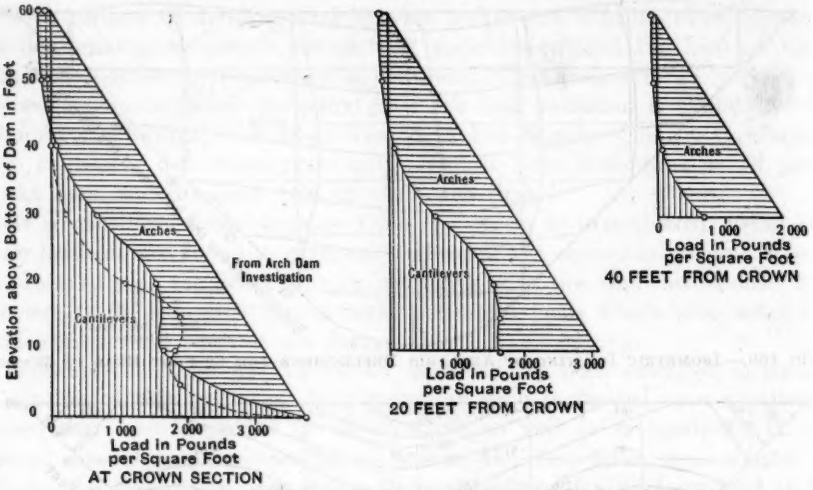

FIG. 172.—LOAD DISTRIBUTION DETERMINED FROM SECOND ANALYSIS.

Fig. 170, which shows horizontal strains, and Fig. 171 which shows vertical strains computed from the second analysis, may likewise be compared with Figs. 94 and 95,‡ and 153 and 154.§ It is evident that the strains are in considerably closer agreement than the deflections.

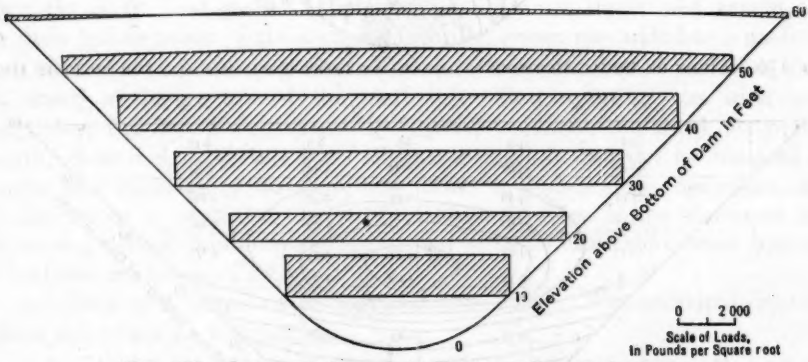

FIG. 173.—LOADS ON ARCHES AS FOUND IN SECOND ANALYSIS.

Fig. 172 shows the load distribution from the second analysis, on three vertical sections. It is seen that the variation in load near the crest is not large in magnitude, but that it has a great effect on the deflection of the slender arch sections. Fig. 173 shows the loads on the arches.

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 133.
† *Loc. cit.*, p. 224.
‡ *Loc. cit.*, p. 138.
§ *Loc. cit.*, p. 229.

The crest deflection curves have not been duplicated in magnitude by any one of the four analyses, although good agreement has been achieved by some analyses at all lower elevations. The crest deflection curve from the celluloid model requires that the arch carry about 330 lb. per sq. ft. by direct thrust. The highest cantilever reaction which it was possible to use and get deflection agreement was 190 lb. per sq. ft. at the crown and much less at other points on the crest; the load carried by direct thrust in the arch was no more than 120 lb. per sq. ft.

J. L. SAVAGE,* AND IVAN E. HOUK,† MEMBERS, AM. SOC. C. E. (by letter).‡—The Committee has furnished a great mass of valuable technical information regarding arch dams and related problems. The comprehensive experimental investigations at the Stevenson Creek Test Dam, together with the elaborate mathematical analysis of the data, will prove of great value to all engineers engaged on arch dam design. Moreover, the painstaking laboratory studies of the physical properties of concrete will prove of great value to all engineers engaged on the design and construction of concrete works, even though the works include no adaptation of arch principles.

Questions that naturally arise in studying the experimental data obtained at the Stevenson Creek Test Dam are: (1) How can the information best be utilized in the design of arch dams? (2) What method of design agrees best with the analysis of the test data? and (3) How should the most approved methods of analyzing arch dams be modified in order to secure more accurate information regarding location and magnitude of maximum stresses to be expected in the finished structure? Unfortunately, it was not possible for the Committee to include definite recommendations regarding methods and principles of arch dam design in its preliminary report because of the unfinished nature of the investigations. However, a brief treatment of such questions was presented,§ and certain conclusions stated thereon are worthy of emphasis, especially those relating to the time-worn cylinder formula and the theory of uniform arch loads. Regarding the former, Professor Slater states:‖ "It is quite obvious that the cylinder formula is entirely inadequate to represent conditions found in an elastic arch of this kind." Regarding the latter, he states:‖

"Therefore, for the portion of the dam at and above the 40-ft. elevation, the assumption of an elastic arch with uniform radial loading did not give stresses sufficiently close to those found from the test to warrant this assumption as a basis for design."

In order to check the methods used in designing important Reclamation dams, the U. S. Bureau of Reclamation has made a study of the Stevenson Creek Test Dam data by the trial-load method of analysis—the method which has been developed in the Denver Office of the Bureau during the past five years.¶ Generally speaking, this method, as now used, assumes that no load

* Chf. Designing Engr., U. S. Bureau of Reclamation, Denver, Colo.

† Research Engineer, U. S. Bureau of Reclamation, Denver, Colo.

‡ Received by the Secretary, May 11, 1929.

§ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, pp. 187–190.

‖ *Loc. cit.*, p. 189.

¶ "Analysis of Arch Dams by the Trial Load Method," by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 61; also discussion by J. L. Savage and Ivan E. Houk, Members, Am. Soc. C. E., *Loc cit.*, October, 1928, Papers and Discussions, p. 2317.

is carried by tension forces, but takes into account many effects not heretofore considered in the study of arch dams. For instance, in the latest use of the trial-load method the analysis has taken into account uplift at the base of the dam and within the concrete at all horizontal sections; the reduced effect of cantilever action, due to considering the sides of the cantilevers as radial planes instead of parallel planes; the effect of temperature changes in the concrete, due to both setting heat and atmospheric temperature variations; the effect of tangential shear between arches; the effect of twist; and the effect of foundation and abutment deformation in both cantilever and arch elements.

In the trial-load analysis of the Stevenson Creek Test Dam, the results of which are shown in Figs. 174 to 181, inclusive, the effects of twist, tangential shear, and temperature variations in concrete were not considered. The latter were omitted, because the actual tests on the dam were made in such a way as to practically eliminate temperature effects. Methods for considering the effects of twist and tangential shear had not been developed at the time the trial-load analysis of the Stevenson Creek Test Dam was made. However, it is not likely that consideration of these effects would materially alter the final results in the case of a dam of the size and shape of the Stevenson Creek Test Dam. The analysis of the Test Dam data showed that, in all cases, the sum of the loads carried by horizontal bending, direct arch thrust, and vertical bending was very nearly equal to the total water pressure. Consequently, the transmission of load by tangential shear and twist could not have been important.

Uplift pressures on the base of the dam and in the concrete, in the trial-load calculations, were assumed to act over one-half the areas of the horizontal sections, and to vary as straight lines from full reservoir pressures at the up-stream face of the dam to zero at the down-stream face. A crack was assumed to form at the up-stream edge of the base in the central part of the dam, as was found to occur in the tests, and tension was assumed to act in the other parts of the dam wherever indicated by the calculations. No vertical cracking was assumed to take place in the upper and lower parts of the crown section, such as that in the dam, and no cracking was assumed at the abutment ends of the lower arches. As regards the latter assumption, the arch computations are on a slightly different basis than the cantilever calculations, since the cantilevers are assumed to be cracked at the base while the arches are not assumed to be cracked at the ends. However, it is only at the lower elevations that horizontal tension occurs on the up-stream face at the abutments, and for these arches the increased deflection obtained through the calculation of cracked arches would be relatively small. Incidentally, the increased deflections thus obtained would result in a closer agreement between the calculated and observed deflections.

In the trial-load analysis, made by A. W. Simonds, Assoc. M. Am. Soc. C. E., and Associate Engineer in the Denver Office of the Bureau of Reclamation, arch action was assumed to begin as soon as the reservoir started to fill, so that the calculated arch deflections are measured from the up-stream position of the cantilevers as calculated for an empty reservoir. The modulus

of elasticity was assumed to be: (*a*) For the concrete, 3 600 000 lb. per sq. in.; and (*b*) for the granite foundation and abutments, 2 000 000 lb. per sq. in., as determined by the tests of concrete and granite samples described* in the report of the Committee. The unit weight of the concrete was assumed to be 150 lb. per cu. ft. The trial-load studies were confined to the action of the dam under the full-load tests, no calculations being made for loads exerted when the reservoir was partly full.

Fig. 174 shows the load distribution, the up-stream cantilever deflection with reservoir empty, and the deflections of both arch and cantilever elements under full load, all as determined by the trial-load calculations. Although the load distribution curves would indicate a comparatively uniform distribution between arch and cantilever elements, the small variations on the upper arches, near the abutments, were important as regards stress distribution as well as agreement of arch and cantilever deflections. Below Elevation 40 the loads on the arches were uniform from crown to abutment. A comparison of the load

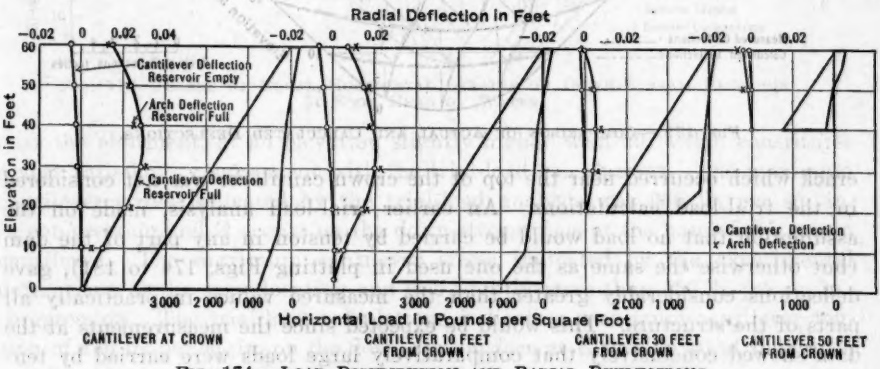
Radial Deflection in Feet
Elevation in Feet
Cantilever Deflection Reservoir Empty
Arch Deflection Reservoir Full
Cantilever Deflection Reservoir Full
° Cantilever Deflection
× Arch Deflection
Horizontal Load in Pounds per Square Foot
CANTILEVER AT CROWN
CANTILEVER 10 FEET FROM CROWN
CANTILEVER 30 FEET FROM CROWN
CANTILEVER 50 FEET FROM CROWN

FIG. 174.—LOAD DISTRIBUTION AND RADIAL DEFLECTIONS.

distribution curves with those determined from measurements at the dam (shown at the right of the diagrams in Fig. 126†) illustrates that the trial-load analysis puts more load on the arches and less on the cantilevers in the lower half of the dam; more negative load on the cantilevers and more positive load on the arch at the top of the dam; and does not alter materially the load distribution in the other parts of the structure. The comparatively small differences between the locations of the circles and crosses on the deflection curves in Fig. 174 indicate the agreement of the cantilever and arch deflections as calculated by the trial-load method.

Fig. 175 is an isometric diagram of deflections for the 60-ft. head of water, showing the comparison between the deflections determined by the trial-load analysis, and those measured at the dam during Tests 11, 12, and 13, the curves for the latter being copies from Fig. 86.‡ In the computations, the following assumptions were made: (*a*) Concrete takes tension; (*b*) a crack occurs at the base of the cantilevers; (*c*) there is a yielding of foundation and abut-

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, pp. 155–156.
† *Loc. cit.*, p. 174.
‡ *Loc. cit.*, p. 133.

ments; (*d*) the value of E for concrete equals 3 600 000; and (*e*) the value of E for the granite abutments equals 2 000 000 lb. per sq. in. It will be noticed that the calculated deflections are less than the measured deflections at practically all locations, the differences being greatest near the crown of the top arch. Undoubtedly, these differences are due to the fact that the vertical

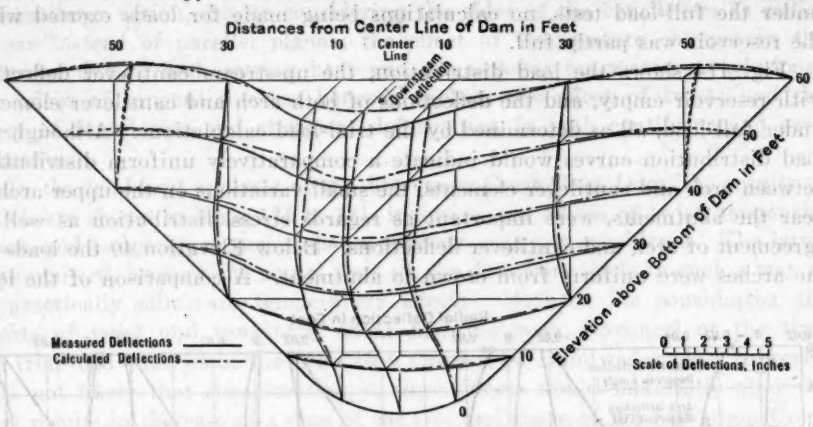

FIG. 175.—COMPARISON OF ACTUAL AND CALCULATED DEFLECTIONS.

crack which occurred near the top of the crown cantilever was not considered in the trial-load calculations. An earlier trial-load analysis, made on the assumption that no load would be carried by tension in any part of the dam (but otherwise the same as the one used in platting Figs. 174 to 181), gave deflections considerably greater than the measured values in practically all parts of the structure. This would be expected since the measurements at the dam showed conclusively that comparatively large loads were carried by tension forces in many parts of the structure, even though cracks did form at some locations as a result of unusually high tension stresses.

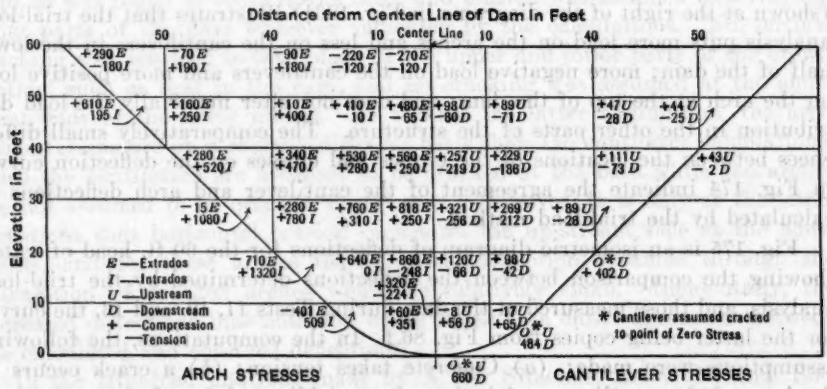

FIG. 176.—DEVELOPED ELEVATION.

Fig. 176 shows the stresses at the up-stream and down-stream sides of the dam, as determined by the trial-load analysis, arch stresses being shown at the

left of the diagram and cantilever stresses at the right. It will be noticed that the maximum calculated arch stresses occur at the abutments of the arch at Elevation 20, the stress at the down-stream side being 1 320 lb. per sq. in. compression and that at the up-stream side, 710 lb. per sq. in. tension. The experimental data obtained at the dam indicated a maximum arch stress of approximately 1 100 lb. per sq. in. in compression on the down-stream face

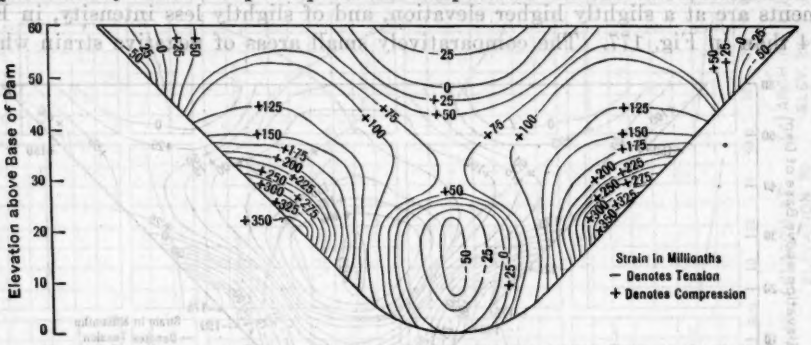

FIG. 177.—LINES OF EQUAL HORIZONTAL STRAINS ON DOWN-STREAM FACE FOR 60-FOOT HEAD OF WATER.

near the abutment, at an elevation slightly higher than 20, which constitutes a very satisfactory agreement with the trial-load calculations. The maximum cantilever stress indicated by the trial-load analysis was 660 lb. per sq. in. in compression and it occurs on the down-stream face at the base of the crown cantilever. The maximum cantilever stress indicated by the experimental data occurs at the same location and was slightly more than 450 lb. per sq. in., compression. The trial-load calculations indicate a maximum cantilever tension of 256 lb. per sq. in. on the down-stream face at Elevation 30 of the crown cantilever, where the experimental data indicate a tensile stress somewhere between 180 and 270 lb. per sq. in.

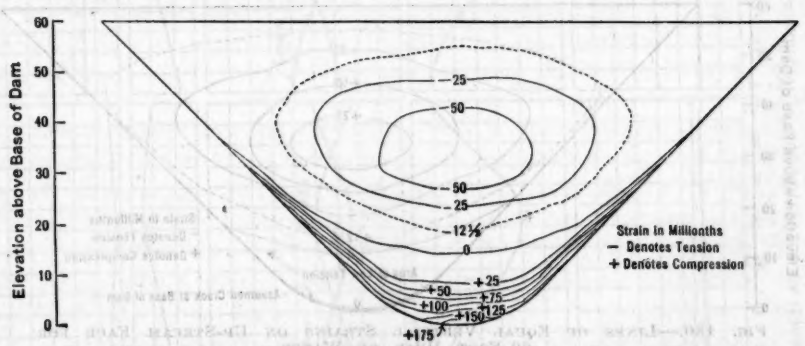

FIG. 178.—LINES OF EQUAL VERTICAL STRAIN ON DOWN-STREAM FACE FOR 60-FOOT HEAD OF WATER.

Figs. 177 and 178 showing lines of equal horizontal and vertical strains on the down-stream side of the dam, as determined by the trial-load analysis, were prepared for comparison with Figs. 94 and 95.* A study of Figs. 177 and 94,

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 138.

showing lines of equal horizontal strains, indicates an interesting similarity between the calculated and experimental data, especially in the left half of the diagrams. Fig. 177, being based on calculated data, is symmetrical about the crown section, while Fig. 94, being based on experimental data, is not entirely symmetrical. This lack of symmetry in the observational data is discussed in the report.* The areas of maximum positive strain near the abutments are at a slightly higher elevation, and of slightly less intensity, in Fig. 94 than in Fig. 177. The comparatively small areas of negative strain which

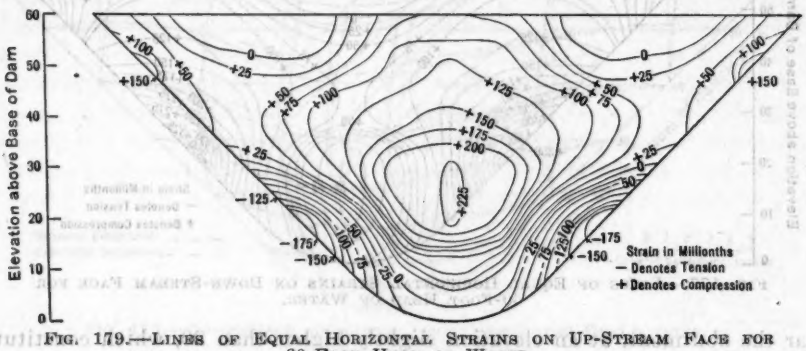

FIG. 179.—LINES OF EQUAL HORIZONTAL STRAINS ON UP-STREAM FACE FOR 60-FOOT HEAD OF WATER.

occur near the abutments of the upper arches in Fig. 177 are entirely absent in Fig. 94, probably as a result of the vertical cracking at the crown section which was not considered in the trial-load calculations. Lines of positive strain equal to 0.000075 cross the crown section horizontally, at elevations of about 30 and 40, in Fig. 94, but do not cross at all in Fig. 177, although they probably would have crossed between Elevations 30 and 40 if a horizontal section midway between these elevations had been considered in the calculations.

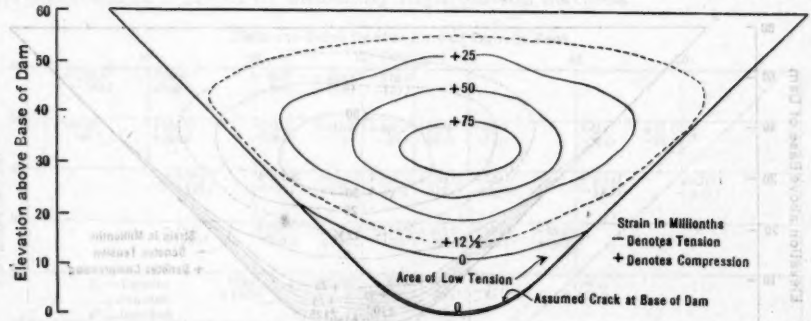
Elevation above Base of Dam
Strain in Millionths
— Denotes Tension
+ Denotes Compression
Area of Low Tension
Assumed Crack at Base of Dam

FIG. 180.—LINES OF EQUAL VERTICAL STRAINS ON UP-STREAM FACE FOR 60-FOOT HEAD OF WATER.

Figs. 178 and 95 do not show as great a similarity as Figs. 177 and 94. However, a study of the way in which the lines of equal strain cross the crown section shows a satisfactory agreement. Moreover, the areas and intensities

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 130.

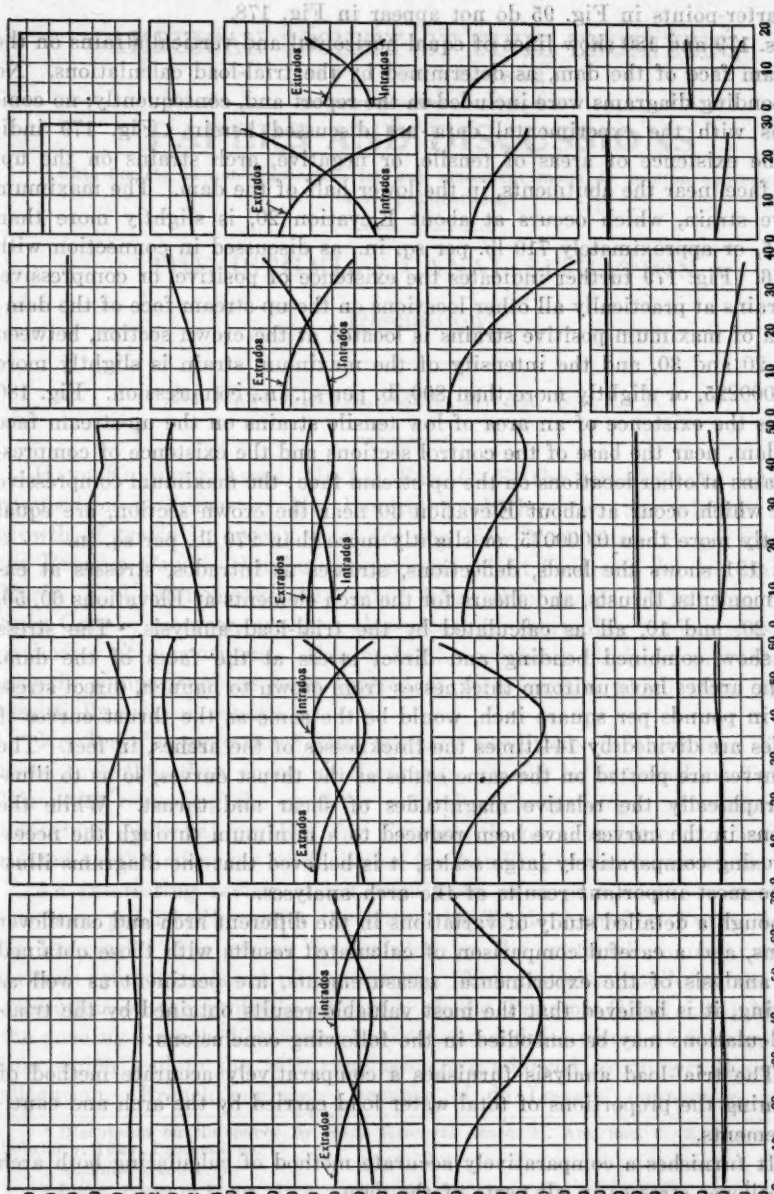
Load in Pounds
Radial Deflection in Ft.
Stress in Pounds per Square Inch Compression
Stress in Pounds per Square Inch Tension
Moment in Thousands of Foot-Pounds
Thrust in Thousands of Pounds
Shear in Thousands of Pounds
Extrados
Intrados
Distance from Center Line of Dam in Feet
(a) ARCH AT ELEVATION 60
(b) ARCH AT ELEVATION 50
(c) ARCH AT ELEVATION 40
(d) ARCH AT ELEVATION 30
(e) ARCH AT ELEV. 20
(f) ARCH AT ELEV. 10

FIG. 181.—ANALYSIS OF ARCH ELEMENTS OF STEVENSON CREEK ARCH DAM: TRIAL LOAD METHOD.

of the positive strains near the base of the dam are practically the same in the two diagrams. The areas of negative strain of from 0.000075 to 0.000100 near the quarter-points in Fig. 95 do not appear in Fig. 178.

Figs. 179 and 180 show lines of equal horizontal and vertical strains on the up-stream face of the dam, as determined by the trial-load calculations. No corresponding diagrams were included in the report and, consequently, no comparisons with the experimental data are discussed herein. Fig. 179 indicates the existence of areas of tensile, or negative, arch strains on the up-stream face, near the abutments, in the lower half of the dam. The maximum negative strain, which occurs at about Elevation 20, is slightly more than 0.000175, or approximately 710 lb. per sq. in., as discussed in connection with Fig. 176. Fig. 179 further indicates the existence of positive, or compressive, arch strains at practically all other locations on the up-stream face of the dam; the area of maximum positive strains is located at the crown section, between Arches 20 and 30, and the intensity of the maximum strain is slightly more than 0.000225, or slightly more than 800 lb. per sq. in., compression. Fig. 180 indicates the existence of an area of low tensile strains on the up-stream face of the dam, near the base of the control sections and the existence of compressive strains at other locations on the up-stream face; the maximum compressive strains, which occur at about Elevation 30 near the crown section, are equal to slightly more than 0.000075, or slightly more than 270 lb. per sq. in.

Fig. 181 shows the loads, deflections, stresses at intrados, stresses at extrados, moments, thrusts, and shears for the arch elements at Elevations 60, 50, 40, 30, 20, and 10, all as calculated by the trial-load analysis. The stress curves show combined bending and direct stress at the faces of the dam. Since the arches have uniform thicknesses from crown to haunch, direct stress curves, in pounds per square inch, would be the same as the thrust curves if the scales are divided by 144 times the thicknesses of the arches, in feet. The shear curves are plotted on the same scales as the thrust curves, so as to illustrate graphically the relative magnitudes of shear and thrust. While the variations in the curves have been reduced to a minimum through the necessity of using comparatively large scales, it is believed that the diagrams illustrate the most important results of the arch analyses.

Although a detailed study of variations in the different arch and cantilever functions, and a careful comparison of calculated results with those obtained by the analysis of the experimental measurements, are pertinent as well as interesting, it is believed that the most valuable results obtained by the trial-load calculations may be embodied in the following conclusions:

1.—The trial-load analysis furnishes a comparatively accurate method of determining the proportions of total water load carried by the arch and cantilever elements.

2.—It furnishes a comparatively accurate method of calculating both arch and cantilever stresses in all parts of the dam.

3.—The maximum arch and cantilever stresses determined by the trial-load method of analysis are slightly higher than those actually occurring in the dam, so that the use of the trial-load method is on the side of safety without being unreasonably conservative.

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PAPERS AND DISCUSSIONS

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THE ISTHMIAN CANAL SITUATION

Discussion*

By Hans Kramer, Assoc. M. Am. Soc. C. E.†

Hans Kramer,‡ Assoc. M. Am. Soc. C. E. (by letter).§—An interesting comparison is drawn by Mr. Landreth‖ which shows the relation between growth of traffic in recent years for the New York State Barge Canal and the Panama Canal. Similar comparisons might be drawn for the Sault Ste. Marie, Kiel, Manchester, and other large ship canals. The Panama and Suez Canals, however, are in a class apart in shortening trade routes by the perimeter of a continent. Hence, in the economic study of probable traffic growth, comparison has been limited to these canals, which alone represent essentially similar conditions.

Mr. Icaza's suggestion,¶ that sanitary conditions in the Canal Zone are more favorable than those in Nicaragua, should be included in Table 3.** His point regarding the need for a fixed highway bridge, although it is an important National problem for Panama, is not considered to be directly related to the adequacy of facilities for handling water-borne traffic.

An interesting review and a translation of a part of the writer's paper has been published†† under the title, "Le Trafic Actuel et Futur du Canal de Panama". The review is prefaced by the editorial comment that the writer's conclusion is entirely optimistic. This criticism applies to the writer's prediction of the probable adequacy of the Panama Canal until 1970. In the ensuing French discussion, however, no definite errors of fact or assumption which lead to the 1970 conclusion are adduced to support the sweeping allegation. French engineers, because of de Lesseps' connection with the

* Discussion on the paper by Hans Kramer, Assoc. M. Am. Soc. C. E., continued from February, 1929, *Proceedings*.

† Author's closure.

‡ Lieut., Corps of Engrs., U. S. A., Fort Humphreys, Va.

§ Received by the Secretary, April 25, 1929.

‖ *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 499.

¶ *Loc. cit.*, p. 501.

** *Loc. cit.*, August, 1928, Papers and Discussions, p. 1703.

†† *Le Génie Civil*, October 20, 1928.

Panama Canal project, maintain a natural interest in the canal's present and future. A more constructive criticism from that source, therefore, would have been welcomed.

Since the preparation of the paper other reliable opinions have been recorded which tend to bear out the writer's conclusion that the rate of increase in traffic will be more moderate in the future. In testifying before a Sub-Committee of the House Committee on Appropriations on December 11, 1928, Harry Burgess, Colonel, Corps of Engineers, U. S. A., M. Am. Soc. C. E., Governor of the Canal Zone, stated "As nearly as we can tell, the increase will not be more than ten million tons per decade". This forecast is in line with the "constant annual increment" assumption on which Curve *F*, Fig. 2,* is based, and is even more conservative than the 12 500 000-ton increase per decade predicted by the curve.

Similar non-alarming conclusions are reached by the Standing Committee on "Maximum Capacity of Panama and Steps Being Taken by Those in Authority for Enlargement of the Canal for Shipping in the Future". This Committee reported† to the Association of Pacific and Far East Ports at its Fifteenth Annual Convention at Los Angeles, Calif., June 28, 1928: (1) That the canal was then being used to only one-third its capacity; (2) that the normal increase was less than 2% per year; (3) that, therefore, the canal would not be taxed to full capacity until about 1958 or later; (4) that the public should trust the Canal Administration to anticipate the need of enlargement or of building another canal; and (5) that the need for such expansion was assuredly far in the future.

The reasonableness of these viewpoints is supported by the actual Panama Canal record of transits for the calendar year, 1928. During this last year the net tonnage transmitted was 28 943 437, an increase of 332 453 tons, or 1.2% more than 1927 and an average increase of 1 553 598 tons per year, or 6.0% per year for the two-year period since 1926. The ordinate for the 1928 abscissa on Curve *F*, Fig. 2, predicts a probable traffic of 28 300 000 tons, which is within 2.1% of the actual record for the latest year.

After considerable debate in both Houses, the Seventieth Congress passed Senate Joint Resolution 117 which was approved March 2, 1929,

"* * * authorizing an investigation and survey for the purpose of ascertaining the practicability and the approximate cost of constructing and maintaining additional locks and other facilities at the Panama Canal, and for the purpose of ascertaining the practicability and probable cost of constructing and maintaining an interoceanic ship canal across the Republic of Nicaragua".

The Chief of Engineers, under the direction of the Secretary of War, is now formulating a policy for the execution of this mission. The forthcoming study should satisfy legislators, agitators, and the general public that there are no serious economic or engineering causes for alarm in the present Isthmian Canal situation.

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1700.
† *World Ports*, September, 1928, p. 983.

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PAPERS AND DISCUSSIONS

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THE RELATION BETWEEN EARTHQUAKES AND ENGINEERING SUBSTRUCTURES

Discussion*

By Messrs. A. Floris and Norman W. Kelch.

A. Floris,† Esq. (by letter).‡—This paper deals briefly with earthquakes and their effects on structures. To a large extent the material presented is given by the author in his numerous articles previously published elsewhere.§ Furthermore, it contains considerable material which belongs rather to the science of seismology; it has only a comparatively small bearing on structural engineering.

The discussion of the various kinds of waves transmitted to all parts of the earth during an earthquake disturbance, although of extreme value to seismologists and geo-physicists, is certainly of secondary importance to structural engineers. In a homogeneous isotropic body, if only central forces are acting between its molecules, theory indicates that the ratio of the velocities of the longitudinal to the transverse waves is equal to $\sqrt{3}$.‖; or, in other words, the longitudinal waves are 1.73 times as fast as the transverse waves. Since the longitudinal waves have a considerable speed of propagation (from 2 to 6 km. per sec.) it follows that these waves will be fully separated from the transverse waves only in the case of a distant earthquake; that is, when the origin of the seismic disturbance is at a considerable distance from the seismograph

* Discussion of the paper by Henry D. Dewell, M. Am. Soc. C. E., continued from March, 1929, *Proceedings*.

† Civ. Engr., Los Angeles, Calif.

‡ Received by the Secretary, April 26, 1929.

§ "Building Against Earthquakes," *Buildings*, February, 1922; "Earthquake Damage to Santa Barbara Buildings," *Engineering News-Record*, July 9, 1925; "Building for Safety Against Earthquakes as Evidenced by the Santa Barbara Earthquake," *Bulletin*, Allied Architects' Assoc. of Los Angeles, August 1, 1925; Report of the Committee on Building for Safety Against Earthquakes, *Bulletin*, Seismological Soc. of America, September, 1925; "Earthquake Damage to Buildings," by Henry D. Dewell and Bailey Willis, *Loc. cit.*, December, 1925; "Earthquake Expectancy in California," *Engineering News-Record*, January 21, 1926, and "Earthquake-Resistant Construction," *Loc. cit.*, April 26 and May 3, 1928.

‖ "Elements of Geophysics," by Dr. Richard Ambronn, tr. from the German by M. C. Cobb, N. Y., 1928, p. 206.

where these waves are recorded. Such earthquakes yield seismograms of great value to the study of the earth's interior, but they are comparatively useless to the structural engineer.

In earthquakes originating near-by, the motion imparted to the seismograph begins with very nearly, if not quite, its full amplitude. In these earthquakes both kinds of waves appear almost instantaneously, since the origin of the earthquake, that is, the seismic focus, is usually not far from 60 to 30 km.* (measured in a vertical direction) from the locality of greatest intensity. Hence, an earthquake that is disastrous to structures often produces the same effect as impact, and its energy dies out quickly after a few oscillations. Structures in earthquake areas must be able to withstand such strong shocks. These impulsive forces can be resolved into a horizontal and a vertical component. Only the horizontal component is of importance in structures, and that may be assumed to act statically for the reasons stated previously. It can be easily determined by the aid of Newton's second law of motion, provided the acceleration of the earthquake movement is known. This condition must be satisfied by each structure if it is expected to resist earthquakes.

There is also one important phenomenon that the author has scarcely mentioned. High, slender structures can fail during seismic oscillations of long duration, but of less intensity, leaving other structures undamaged. This dynamic cause of failure is produced by resonance that occurs when the natural or free period of vibration of the structure coincides with the period of the disturbing extraneous force or, what is the same thing, when the transient frequency of vibration disappears and the structure reaches the steady frequency of the impressed force. It is clear, therefore, that if structures are to be safe against earthquakes they must be able to resist horizontal impulsive forces as well as strains caused by vibrations. For this reason an investigation of the seismic stability of structures must include both cases, if necessary.

In discussing the merits of various structures, the author deals extensively with the safety of dams and with their probable resistance to such forces. His statements are to some extent surprisingly bold. Obviously, they contradict some widely accepted views that are based on the logical conclusions of scientific principles.

For example, the preference given by the author to earth-fill dams as regards their resistance to earthquakes is rather unjustifiable. The fact that those which were located on or adjacent to the San Andreas fault line behaved remarkably well during the San Francisco earthquake of 1906, is not conclusive proof of their reliability under similar circumstances elsewhere.

It is well-known that earthquake waves in an alluvial soil are of great amplitude and very low speed; but, as is to be expected, they are also extremely effective in damaging structures. Earth-fill dams behave very much the same as these soils, and their fate, therefore, is very problematical at the time of a crucial test. The earth particles of these dams are held together by cohesion and friction alone, thus offering a slight resistance to extraneous disturbing forces.

* "A Manual of Seismology," by Charles Davison, Cambridge, 1921, p. 132.

Structures that are more homogeneous and rigid have a far greater chance of withstanding the destructive effects of seismic oscillations. Concrete dams of the gravity, arch, multiple-arch, and Ambursen types belong in this category, and, consequently, from the point of view of seismic resistance, they are superior to earth-fill and rock-fill dams. However, before statements are made relative to the merits of these rigid types to withstand earthquake shocks, it is essential that the peculiarities inherent in each type be investigated. Their static as well as their dynamic behavior during such disturbances must be taken into account at any rate. This is not done by the author, and the result of a casual occurrence pertaining rather to a particular case does not carry much weight in an attempt to classify dams according to their probable resistance to earthquakes. It is decidedly dangerous to draw general conclusions from a special case, as this might mislead those who are not conversant with the actual facts concerning failure.

Undoubtedly, a study of the resistance of dams to earthquakes is of extreme practical importance, despite the fact that it offers great mathematical difficulties. Nevertheless, with certain simplifying assumptions it is possible, by the aid of theory, to gain some knowledge of the stresses caused by seismic forces in dams.

NORMAN W. KELCH,* ASSOC. M. AM. SOC. C. E. (by letter).†—The author's remarks‡ on the value of deep foundations for protection against earthquake cannot be emphasized too strongly, as this fact has been borne out conclusively by experience. Most larger buildings are provided with basement garages and many with sub-basements below this. There are several buildings in Los Angeles, Calif., having four stories of basement, with foundations that extend to at least 60 ft. below the street level. It follows definitely that such modern buildings are subjected to far less earthquake stress than those erected formerly which had shallow basements and shallow foundations. It would seem quite feasible that, in calculating earthquake stresses, Mr. Dewell's reference to the value of deep foundations be well considered as materially decreasing the actual stress on the superstructure. When the foundations are set on firm, solid ground, it is not quite clear why Mr. Dewell would wish to have them interconnected with reinforced concrete. The earthquake force acts on the entire area simultaneously and there is no effort to separate the columns. If the building synchronized with the earthquake force and set up a swaying motion, then there would be excessive thrusts on the footings, alternating from one side of the building to the opposite side. To avoid the settling of foundations under this action perhaps it would be better to make all exterior wall column footings as combined footings with the next tier inside or carried back to a point considered safe against such action. A heavy reinforced concrete tie, as Mr. Dewell suggests, would seem only effective in direct compression or bending and would not act as a tensile member in holding the columns together during an earthquake causing non-uniform distribution

* Engr., Clay Products Inst. of California, Los Angeles, Calif.
† Received by the Secretary, March 30, 1929.
‡ *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1743.

of loads. Great stress should be laid on Mr. Dewell's suggestion to keep the structure from actual contact with the upper or surface ground, except by slip-joints, since considerable damage could be wrought through this agency as well as through low-storied buildings adjacent to higher ones.

Mr. Dewell's observation of the concrete building foundations in Santa Barbara, Calif., which were sheared because the cold construction joints provided planes of weakness, brings up a serious problem in connection with the superstructures as well as the foundation. It is well known and demonstrated that the destructive earthquake force is one inducing horizontal shear. The force is distributed at each floor through the floor system and acts on the vertical sections of the building either on the columns or on a combination of columns and walls. The point of maximum shear is at the tops and bottoms of the vertical sections and in reinforced concrete construction there is always a "cold" joint at the tops and bottoms of columns and walls. The joint is at the bottom because of necessity in construction and at the top because of the necessity of allowing for vertical shrinkage before applying the floor systems. There have been failures of reinforced concrete buildings because of this weakness in shear at these points and considerable thought and attention should be directed to this element if designers are to demand that reinforced concrete structures shall resist earthquake shear effectively.

While Mr. Dewell is correct in stating* that earthquake damage in the case of pipe lines is largely a question of location, and that pipe lines crossing a fault slip will be ruptured, nevertheless experience does not bear out his contention as applying "particularly to lines of vitrified pipe". At the time of the Santa Barbara earthquake, of 1906, the only sewers which were seriously affected by the earthquake were the main outfalls, 42-in. pre-cast concrete pipe sewers, and 14-in. and 16-in. vitrified clay pipe, both being under construction and in the hands of contractors at that time.

The concrete pipe was supported on timber saddles; the trenches had not been back-filled, and were located in a direction parallel to that of the surface waves. Since they were free to act in the open, the joints parted, but the reinforced concrete pipe itself was not materially damaged. The contractors relined the pipe by cutting down the high places and then re-caulked the pipe joints.

The clay pipe was laid in the sand ground of the water-front, where there was heavy circulation of ground-water. It was located at right angles to the direction of the surface waves, so that the entire length of the pipe was lifted and dropped and thrown from side to side, with the rise and fall and lateral movements of the earthquake. Since this line was on the ocean-front highway, the city authorities had insisted on the pipe being back-filled as soon as it was laid, and this back-fill had not settled. At the time of the earthquake this loose fill brought about a bumping action and damaged some of the pipe bells; and since the job had not been paid for, the municipal authorities insisted that new pipe be furnished.

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1742.

The remainder of the Santa Barbara sewer system, which is entirely of clay pipe, with the exception of the large pipe used in the outfall, was practically undamaged. The only difference noted after the earthquake was that, at the time of heavy rains, there was some increase in surface water infiltration. Present specifications provide cement mortar joints using 1 part cement and 1 part sand, and most of the sewers were laid before the present ordinances requiring straight cement mortar were adopted. The Assistant City Engineer reports that he had personally inspected all the brick manholes in the city after the earthquake and found them in perfect condition.

Sewer pipe, being in a class of thin-walled structures, would naturally suffer more than more massive types, but this applies to concrete as well as to clay. As both clay and concrete are required to pass the same strength tests there should not be—and experience demonstrates that there is not—any advantage in concrete over clay. As a matter of fact, since the main trouble is at the joints, it is difficult to understand why Mr. Dewell states that reinforced concrete would offer any advantages over vitrified pipe.*

In considering the movement of the sub-surface due to earthquake it would seem that sectional pipe lines have certain advantages over monolithic lines and that a sectional pipe line with elastic joints would be the ideal. The sectional pipe has telescoping bell-and-spigot joints laid tight so as to overlap in the direction of the flow, pipe against pipe. The jointing material is placed outside the effective shell thickness of the pipe simply as a seal. If this compound is of cement and provides a "cold" joint which, as Mr. Dewell points out,† will not stand earthquake stress, then the seal is broken, but the line of pipe remains in effective service.

It would seem that an earthquake movement of a monolithic pipe line would cause rupture directly through the shell of the line, seriously damaging its service by allowing the escape of sewage during dry seasons and water and sand infiltration during wet seasons. It is observed that the present tendency in many important sewer systems is to use vitrified clay pipe with a special asphaltum jointing compound which provides a seal against considerable internal water pressure and permits earth movement instead of endeavoring to resist it.

It would be enlightening to test some sections of reinforced concrete sewer pipe which has been in use for 10 or 20 years to note the effects of use and location on the concrete and the reinforcing regarding both strength and imperviousness.

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* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1742.

† *Loc. cit.*, p. 1743.

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PAPERS AND DISCUSSIONS

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REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION ON "A NATIONAL RECLAMATION POLICY"

Discussion*

BY MESSRS. GEORGE W. FULLER, W. F. RUSSELL, NILS A. OLSEN, AND L. C. GRAY.

GEORGE W. FULLER,† M. AM. SOC. C. E.—This report is of great interest to all who have occasion to consider present or prospective reclamation projects. Obviously, there is more irrigated land than is now needed to supply farm products to the population of Continental United States. Unless a tariff schedule can be adopted with other arrangements such as to promote the successful sale of farm products abroad, the future of irrigated lands in the Continental United States will depend largely upon the relation between supply and demand as affected by increases in population. Therefore, care should be exercised to guard against the development of irrigation projects tending to cause a continuance of excess production of farm products as compared with market demands.

It is doubtful whether American citizens in general realize how this question of excess production is being affected by a marked decrease in the rate of population growth in the Continental United States. Statistics bearing on this subject have been analyzed by many writers and are well summarized by Dublin.‡ Since the first Federal census in 1790, the United States has doubled in population on an average once in 27 years. During the middle of the Nineteenth Century, however, when immigration from Central and Western Europe was abundant and the growth of the colored population in the Southern States was very rapid, it took only 20 years for the population to double.

* This discussion (of the Report of the Committee of the Irrigation Division on A National Reclamation Policy, published in September, 1928, *Proceedings* and presented at the meeting of the Irrigation Division at San Diego, Calif., October 4, 1928), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. (Fuller & McClintock), New York, N. Y.

‡ "Health and Wealth," by Louis I. Dublin, Harper Bros., 1928.

The situation is now changed. Excess of birth rate over death rate according to the 1920 Census records showed an increase in population of about 11 per 1 000, or 1.1% per year. This indicates that it would now take about 60 years for this country to double in population; that is, on the basis of uncorrected birth and death rates, which vital statisticians say should be adjusted so that the influence of existing conditions will have its due *pro rata* effect on birth and death rates for groups of persons of different ages.

The slowing up of the increase in population in this country is related to many factors. Among them may be mentioned the restriction, since the World War, in European immigration and changed viewpoints in the home with its complications as to servant questions and the continuing custom of the native stock to have small families. One of the effects of pre-war unrestricted European immigration followed by present restricted immigration is that there now exists in the United States an unusually large proportion of female population of child-bearing age. Continuance of restricted immigration will prevent increments to this group from such source and hence the group as a whole will become relatively smaller with a lessening birth rate in the future as compared with recent years when the effect of pre-war immigration is still felt.

Without going into great detail it is sufficient to state that when consideration is given to the foregoing item; to the statistics of life expectancy of persons in each group, arranged in ages; and to the effect of present environment with its modern sanitation, preventive medicine, and personal hygiene, reconstructed birth rates and death rates are derived which indicate that it will take this country about 120 years to double in populaion. In fact, there are prominent statisticians who state that the population of this country will become stationary with a maximum about 200 000 000 people by 2100. Others on the contrary predict, on the basis of ultimate possible food production, a maximum of 300 000 000. Whatever may be the precise outcome as to population growths it is certain that there is nothing in sight to indicate that an increased demand for farm products justifies the early development of large new irrigation projects.

W. F. Russell,* Esq. (by letter).†—The engineering problem of reclamation has been well taken care of to the extent that great irrigation systems have been completed; but the building of a system of canals is only the beginning of reclamation. To say that the responsibility of the Government ends with the building of canals may be compared to the manufacturer of an automobile contending that his car was finished without supplying it with wheels. The preparation of the land to receive the water and the placing of settlers to till the soil is the most important part of the reclamation of arid lands. The money invested in dams, reservoirs, and canals is wasted if this other part of the work is not done just as scientifically and well as was the planning and construction of the canals. In most cases the cost per acre for land preparation and settlement will exceed the cost of construction.

* Field Superv., Colonization Dept., Lethbridge Northern Irrig. Dist., Lethbridge, Alberta, Canada.

† Received by the Secretary, June 8, 1929.

The advancement in the scientific pursuits has been very rapid, and irrigation reclamation is no exception. The man who is to succeed on the land, who is to utilize the water that comes through the canals, and who must produce from the land through the proper use of the water, must be a scientist in every sense of the word. He must acquire more scientific knowledge to be a good farmer than to conduct any other business, and it is time that the people of the United States realized not only that farming is the most important of human activities, but that the farmer is entitled to, and should receive, just and fair compensation for his investment, for his labor, and for his responsibility. Too little importance has been given to the farmer by the leading men, both in Government and financial circles.

If the projects that have been built by the Government are to succeed, the Government must carry the responsibility of completing the work, and that cannot be effected until every acre of land under the canal is producing. The settler cannot finance the preparation of the land. It requires a number of years as well as capital expenditure, to develop raw land to its maximum capacity of production. It also requires considerable capital for a farmer to support his family through these years of development.

The writer has found that a settler going on to raw land with $2 000 to $3 000 is financially embarrassed before the land can be developed to a point where it will produce a comfortable living for his family and begin to pay the capital charges against the land. This condition must be overcome by some policy of finance. It may be a loan against the land, or it may be placed as capital cost and be added to the price of the land. It does not matter how it is done. It is the responsibility of the Government to protect the huge investment it now has, and to protect the interest of the settler, that will make the project a success.

The Government policy of financing construction is to be commended, but it did not go far enough. There are those who might say that this would be charity and that, if the Government were to undertake to supervise the development, it would be paternalism. Yet they are willing that supervisory methods be used for the construction of the dams and reservoirs, and the laying out of the canals; furthermore, they are willing that the same methods should be used in the construction of the power plants and lines of distribution; and that other lines of industry should use the same methods. They will agree that it is necessary that a policy of financing must be instituted in order that these industries should succeed; but if the same principles are applied by the Government which has already gone a long way with the work of reclamation after all other agencies would go no farther, it is charity and paternalism.

The problem of proper development of the land already under Government canals and the settlement of the same by a class of people fitted for the work is one that will require more skill and closer supervision than any part of reclamation the Government has ever attempted; and until some policy for successfully financing the unoccupied land is worked out and put in operation, the great reclamation problem will remain unsolved.

In Idaho and Arizona farms that are properly developed, properly equipped, and properly operated, are paying their way. The families on these lands are

enjoying some of the modern conveniences and necessities, and they are contented. Until the unoccupied land is in the same condition, its owners will not be able to pay its portion, and until the proper financing and supervision is available it will continue to be unoccupied.

The class of settler required for this land is no longer willing to undergo the hardships and privations of the old days. Families demand a standard of living in keeping with the times, and if the rural population is to continue to advance in education, etc., and be able to handle its business in competition with the world, there must be modern conveniences on the farm. What manufacturing concern would think of trying to operate without the latest and most efficient machinery and the latest methods to meet competition? Just the same thing should apply to the development of new lands if they are to become a success; modern methods and the best machinery must be used.

The Lethbridge Northern Irrigation District, in Southern Alberta, Canada, would never have been successfully settled had it not been for the policy of supervision and financing put in operation by the Alberta Government. After four years the Government is just beginning to see the results from the application of this policy. It has fully demonstrated, however, that it is "a step in the right direction". In less than four years the district has been fully settled with a desirable class of citizens, most of whom are contented. Fully 75% are in better financial condition than when the project was begun. Most of the settlers placed on the project have stayed. Of 738 families placed, only 26 have moved away. While many problems have been encountered and mistakes have been made, the Alberta experiment has been fully justified. No other agency could have attained the same degree of success.

NILS A. OLSEN,* ESQ. (by letter).†—Certain conclusions embodied in Mr. Lippincott's paper‡ were expressed by Secretary of Agriculture Jardine§ in discussing the significance of the interest exemption on construction charges as a special subsidy to particular locations. He indicated that such a stimulus in principle represents a form of discrimination against established farmers and new settlers elsewhere, and that, in the present period, it is not justified by a need for enlarging the basis for the Nation's supplies of farm products.

At the same time he pointed out, as Mr. Lippincott has done, that reclamation is frequently connected incidentally with other objectives, such as flood control, promotion of navigation, and development of water power. Under these conditions the advisability of a particular project must be determined by consideration of the entire complex of objectives. Even under such conditions, it would be difficult to justify the interest exemption for reclamation.

L. C. GRAY,‖ ESQ. (by letter).¶—In connection with Mr. Lippincott's interesting paper,‡ attention may be called to the views of the late Mr. R. P. Teele,**

* Chf., Bureau of Agricultural Economics, U. S. Dept. of Agriculture, Washington, D. C.

† Received by the Secretary, June 19, 1929.

‡ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1193.

§ Annual Rept., Secretary of Agriculture, 1927, pp. 26–27.

‖ Prin. Economist in Chg., Div. of Land Economics, U. S. Dept. of Agriculture, Bureau of Agricultural Economics, Washington, D. C.

¶ Received by the Secretary, June 19, 1929.

** *Department Bulletin No. 1257*, "Land Reclamation Policies in the United States," also, "The Economics of Land Reclamation."

formerly of the Bureau of Agricultural Economics, U. S. Department of Agriculture. Mr. Teele had made intensive studies of these matters, and his findings, therefore, have an important bearing on the issue.

The Committee has rendered a public service by its analysis of the Federal Reclamation Policy. Mr. Lippincott might have emphasized still further the points that the exemption of interest (1) increases the local pressure to obtain Federal funds for reclamation projects; (2) tends to stimulate the undertaking of projects which would not be justified by prospective farm income if interest were considered; and (3), therefore, imposes a level of capitalization which must be written down to a lower level when the project is acquired by subsequent purchasers after the period of exemption is concluded.

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THE PRACTICAL UTILITY OF HIGHWAY TRANSPORT SURVEYS

Discussion*

By Messrs. Charles Randolph Thomas, Jr., and G. F. Schlesinger.†

Charles Randolph Thomas, Jr.,‡ M. Am. Soc. C. E. (by letter).§—The analysis made by Mr. Schlesinger shows rather clearly the importance of a study of population figures in planning roads. "The volume of highway traffic in an area is largely a function of the population of the area," as stated‖ and proved by the author for Ohio, is also a fundamental proposition laid down by the late A. M. Wellington, M. Am. Soc. C. E., as applied to railroads. The development of modern highway traffic is producing conditions that are more and more like those that apply to railroads and telephone companies. Similar methods of analysis, which have reached a high degree of refinement in railroad and telephone traffic studies, may be applied with some degree of success to an analysis of highway conditions.

For purposes of predicting future traffic on a route, the great value of a traffic survey seems to be that it establishes the fact that highway traffic in general varies with population. In other respects, a traffic survey is like "locking the stable door after the horse is gone." The traffic is already there. Just how much traffic may be expected and what set of conditions will produce a given traffic is the information that the projector of a highway wants.

In cases where the traffic did not seem to vary so exactly with the population on the State roads, the author states¶ that other roads carried the traffic. Might it not also be assumed from the same data that the economic location of the State road was poorly made—the road was not in the right place?

* Discussion of the paper by G. F. Schlesinger, M. Am. Soc. C. E., continued from May, 1929, *Proceedings*.

† Author's closure.

‡ Editor, *Highway Engineer and Contractor*, Chicago, Ill.

§ Received by the Secretary, April 15, 1929. This discussion includes the brief discussion received from Mr. Thomas on October 22, 1928, and published in December, 1928, *Proceedings*, pp. 2762–2763.

‖ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2081.

¶ *Loc. cit.*, p. 2082.

Another interesting conclusion is that only 14% of the traffic exceeded a 50-mile limit of trip*—through traffic is relatively small. An analysis of traffic from the standpoint of the area served would seem to be possible, and such an analysis should give interesting and useful information, especially in predicting traffic.

The determination of the potential traffic in the area would give maximum figures useful in designing. The variable of "attraction of traffic" is eliminated by assuming that the road properly improved will get all the traffic, much of which is now going by other routes. This, of course, applies only to the local 50-mile haul traffic, which in Ohio is 86% of the total.

The subject is one that might be discussed and elaborated at length with interesting results. Suffice it to say that the old railroad principle of location holds true, namely, that there is one best route between two points or a group of points; that this route should be found and improvement made in proportion to traffic to be anticipated when the traffic area is developed to its full present possibilities; and if funds are available it may be wise to anticipate future traffic possibilities of the area.

The economic study of highways as discussed by Mr. Schlesinger also involves: (1) The political and social aspects and the effect of taxation and intangible benefits; (2) the more easily measured costs of construction and maintenance and the adjustment of such costs to individual vehicle operating costs; and (3) the monetary savings or profits, if any, from improved transportation facilities.

By limiting the subject to highways constructed by private enterprise under reasonable charters from the State, many social and political aspects are eliminated. This permits a study of the subject from the standpoint of commercial success or failure as measured by the yardstick of business—money profit or loss.

It is the purpose of this discussion to review briefly the development of old and modern toll roads and to present a method of studying certain of the economic factors to be considered by the engineer in making a survey of a proposed toll-road project under modern conditions, so that he may determine within reasonable limits of error the relative profit or loss of the project.

Toll Roads in the United States and England.—Immediately after the American Revolutionary War there was an insistent demand for improved roads, but the States and counties were too poor and the people would not tolerate excessive taxes for road building. Therefore, the English method of financing and building roads was adopted. The various States granted to private companies the right to build roads and charge tolls for their use.

The fundamental principle of this turnpike system was to transfer the cost of road building from the State to the users of the road. In the beginning only the main roads were to be turnpikes. The weakness of the turnpike system was that there grew up an endless number of isolated companies frequently under-financed and operating in a territory in which the volume of traffic would not permit an adequate return on the investment, even with

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2081.

tolls amounting to all that the traffic would bear. (A similar situation existed later with respect to some railroads). These private turnpikes were so short that the traveler was annoyed continually by the obstruction of gates as he passed from the road of one company to that of another.

Conditions in England under this form of turnpike system have been described by Mr. Edwin A. Pratt.* He states that when the need arose for better roads in any district, a group of land owners and others would apply to Parliament for authority to raise a loan to be used for improvements. They would then build or repair a road and set up gates or bars at which they could exact tolls for the payment of debts and for maintenance charges. In theory, this was intended as a temporary arrangement and that after the first cost was paid the road would become free for public use. Charters were granted for limited periods of about twenty years and these were renewable if the debt remained unpaid at the end of the period. Mr. Pratt states that:

"* * * One result of the excessive localization of the turnpike system was that trusts of absurdly large proportions were created to look after absurdly small stretches of road".

The turnpike system, accordingly both in England and America, fell into disrepute; it was killed by mismanagement and lack of business due to railroad competition before the coming of the revolution in highway transportation which was brought about by the motor vehicle. At the present time there are only a few of the old-fashioned turnpike companies in existence.

Nevertheless, the era of road building, brought on by the introduction of private initiative, marked the beginning of a period of rapid expansion in general business due to improved transportation facilities both in America and England. This was the era of the great engineers Macadam and Telford.

Under modern traffic conditions in which highway traffic is completely motorized, only a few roads have been built by private capital. The failures of the Nineteenth Century are still too fresh in the minds of the people, and there is a widespread prejudice against toll roads, in many cases without sound reasons.

Toll Roads in Italy.—The outstanding modern development in toll highways built to care for motor traffic is the "autostrade" system in Italy. It has been projected and partly developed under the engineering leadership of Piero Puricelli. His basic ideas on projecting these motor roads, as expressed in a communication to the writer, are as follows:†

1.—Motorways should supplement the ordinary highways rather than compete with them, and for this purpose should be designed for the greatest degree of efficiency consistent with the expected traffic.

2.—These roads have a special function and should be constructed only where peculiar local conditions of traffic, etc., indicate their advisability.

3.—Toll financing of motorways is not a characteristic necessity, but it is a legitimate means of obtaining income from a capital expenditure.

* "History of Inland Transportation and Communication in England," by Edwin A. Pratt, p. 82.

† Also, see, "Motor Roads and the Milan Lakes," by Piero Puricelli, Autostrade, Milan.

4.—In their projection, construction, and operation, motorways are comparable to railroads. They should be designed for express traffic in a class distinct from local highways.

5.—Under private enterprise, the selection of routes and the execution of a project are regulated by the presence or absence of factors that insure a satisfactory financial return. Political pressure is reduced. Motorways should be built only where traffic studies give promise of a sufficient income.

6.—Existing roads need not necessarily be abandoned. If they are improved they may serve as feeders to the express highways.

7.—The mere presence of poor roads that are the result of improper maintenance does not justify the construction of a motorway.

8.—Existing roads often do not meet the needs of motorways because they are narrow and winding, they have bad surfaces, sharp turns, concealed intersections, interurban traction lines along the side, grade crossings, and inadequate traffic control. They are inclined to pass frequently through congested areas, and they are subject to the nuisances created by dust. These conditions are most easily controlled by designing separate express motorways. It is estimated that the Italian Government will take over the present motorway system before 1980.

The Motorway from Milan to the Italian Lakes Region.—This 60-mile highway was begun in 1923 and placed in operation in 1925. It is controlled by a limited liability company and cost approximately $75 000 per mile. The obligations were issued by the company and the Government guaranties the payment of interest, in return for which it takes over the control of the road in 1973.

In 1926, the receipts and expenditures were as follows:

Receipts:	
Trip tickets	$185 000
Season tickets	30 000
Advertising space	25 000
Total receipts	$240 000
Expenditures:	
Operation and maintenance	$ 80 000
Sinking fund and loans	72 500
Amortization of capital	12 500
Interest charges	70 000
Total expenditures	$235 000

The balance sheet thus showed a net profit of $5 000. There was an average of 700 vehicles per day of all kinds using the motorway during 1926 and in 1927 the daily average was 900. During the first months of 1928 the average was greatly increased.

Tolls.—The toll schedule in Table 20 was in effect for two years, after which a 20% reduction was adopted. Subscription cars are good for one year over the entire system, and provision is made for companies owning several vehicles; that is, a 20% reduction for two vehicles, 25% for three, and 30% for four or more. Public service motor vehicles are not issued subscription

cards and are charged triple tolls. Vehicles with solid rubber tires pay double toll.

TABLE 20.—TOLLS PAID ON MILAN-LAKES MOTORWAY.

Type of vehicle.	Ticket.				Subscription.		
	Complete Journey.		Short Journey.		Fifty coupons.	Annual.	Half year.	Quarterly.
	Single.	Return.	Single.	Return.				
Small vehicles.....	$0.50	$0.75	$0.30	$0.50	$17.50	$45.00	$38.00	$30.00
Cars and trucks, to 17 h. p. trailers....	0.75	1.00	0.40	0.65	25.00	67.50	56.00	45.00
Cars and trucks 17 to 26 h. p.....	1.00	1.50	0.60	0.90	33.75	90.00	75.00	60.00
Cars and trucks of more than 26 h. p..	1.50	1.90	0.80	1.10	42.50	112.00	94.00	75.00
Buses and cars, to 20 seats.....	2.50	3.75	1.60	1.90	.....	.....	.....	.....
Buses and cars of more than 20 seats.	3.75	5.60	2.40	3.30	.....	.....	.....	.....

Entrepreneur Studies.—The inception of a toll-road project presents certain speculative questions as to the profits possible from the investment of capital. These may be listed somewhat as follows:

1.—Will the financing, construction, maintenance, and operation costs permit income with a sufficient margin to provide for dividends and the retirement of the capital invested?

2.—What are the sources of present and future traffic and how productive of income traffic will be these sources?

3.—What are the details of immediate and future construction and operating expense on the most economical route and just where should this route be located?

4.—What is the proper investment to make so that traffic will be handled most economically and, at the same time, produce the greatest net revenue?

5.—What kind of franchise may be obtained from the State or local authorities and what are the conditions of the franchise, such as duration, recapture, renewal, etc., that must be met?

6.—What measures will be necessary to attract traffic to the highway and what will be the cost of a measure such as advertising?

To answer some of the questions is the mission of the engineer employed to make a preliminary report; the answer to others has an element of speculation which will be discovered only when the highway has been in operation over a period of years.

Scope of Preliminary Engineering Report.—The scope of the preliminary engineering report suggested is limited by the questions asked by the entrepreneurs as to the feasibility of the project, and by the time and funds available for the investigation.

Two basic ideas must be kept in mind in making such a report: First, that a motor vehicle has freedom of movement and will find the economic route that offers convenience, speed, and economy; and, second, that the best

location between maximum traffic sources must be found and there must be placed on that location the type of road (pavement, signaling, crossing, etc.) demanded by traffic, with due regard to future traffic at the end of a term of years.

The report might well include: (*a*) A traffic reconnaissance of the area through which several possible ground locations might be made; (*b*) estimated traffic from various sources on the approximate paper location made from a study of the area, consideration being given to possible alternate routes at certain points; (*c*) an estimate of revenues both present and possible in the future, as indicated by the population and industrial studies made of the area tributary to the route; (*d*) estimates of construction and maintenance costs; (*e*) estimates of operating costs; and (*f*) summary of financial and charter requirements.

Conclusions and recommendations as to the feasibility of the project are well summarized at the beginning of the report.

Traffic Studies of the Area.—The study of traffic within the area tributary to several possible locations in the general direction thought feasible for the projected highway is limited. Mr. Schlesinger has found that, over large areas such as a State or parts of a State, the traffic on the roads varies directly as the population.* The accumulation of traffic on a railroad route was found by Wellington† to vary as the square of the number of traffic sources, and it is believed that for limited areas the traffic may develop in even a larger ratio on highways.

The author states‡ that the ratio of through traffic (trips of more than 50 miles) was approximately 30% in Ohio; that bus traffic was 1.5% of the total passenger traffic; and that trucks were 9.5% of the total traffic on the State system.

In a study of comparative rail and highway tonnage made in Ohio in 1925§ the proportion of motor-truck tonnage decreased with the length of haul, as shown in Table 21. However, with the improvement in motor vehicles and methods of operation the length of profitable haul is gradually increasing.

The factors entering into a determination of the traffic in the area have been summarized elsewhere by the author as follows:‖

"The formulation of a method for approximating the traffic on a given highway without the necessity of a field survey is within the range of possibility, using the data available as a result of transport surveys. It is believed that the problem is capable of solution if proper consideration is given to such factors as:

1.—The tributary area.
2.—Population density.
3.—Motor vehicular registration.
4.—Character of population—urban or rural.
5.—Classification of predominating industry—industrial or agricultural."

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2081.

† "The Economical Theory of Railroad Location," p. 713.

‡ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, pp. 2079, 2081.

§ Traffic Survey by U. S. Bureau of Public Roads and Ohio Highway Department, reported in *Roads and Streets*, October, 1927, p. 427.

‖ Extract from an address before the American Association of State Highway Officials, 1926.

Maximum traffic will, of course, be the sum of the potential traffic of the area and the foreign traffic coming into the area. However, it is scarcely possible that this maximum can ever be attained.

TABLE 21.—PROPORTION OF MOTOR TRUCK AND RAILROAD NET TONNAGE ACCORDING TO LENGTH OF HAUL FOR AVERAGE MONTH, 1925.*

Length of haul, in highway miles.	Motor Truck.		Rail (Carload).		Rail (Less Than Carload).		Total.	
	Tons.	Percentage of total truck and railroad tonnage.	Tons.	Percentage of total truck and railroad tonnage.	Tons.	Percentage of total truck and railroad tonnage.	Tons.	Percentage of total truck and railroad tonnage.
Less than 20	6 091	84.5	1 112	15.4	10	0.1	7 213	100.0
20 to 39	5 973	54.7	4 803	44.0	145	1.3	10 921	100.0
40 to 59	2 299	32.0	4 484	62.4	404	5.6	7 187	100.0
60 to 99	980	24.2	2 409	59.4	663	16.4	4 052	100.0
100 and more	157	2.3	5 280	77.4	1 353	20.3	6 820	100.0

* Based on tonnage between Columbus, Ohio, and thirty cities having rail connections, Report of U. S. Bureau of Public Roads.

Traffic Reconnaissance of the Projected Line.—Traffic studies of the projected location of a route may be conducted in sections covering a width approximately equal to the normal maximum trip of a motor vehicle.

The comparative weight to allow in securing local traffic for the road; the sacrifices of distance necessary in order to reach certain traffic sources that accumulate traffic; how near to run to cities and towns; the distance gains made in by-passes; and the intermediate changes in alignment to shorten the route, are all matters that must be considered as affecting the total volume of traffic. While the traffic on the road is increased by lengthening the line, if that lengthening is too great drivers become discouraged, the road possibly becomes too congested for the service rendered, and traffic will seek new routes. The effect of a secondary road on the traffic on the main road also must be considered.

It should be remembered that no single road is completely independent of other competing roads, and when congestion comes there is a tendency for the traffic to "spill over" to competing routes. However, traffic has a tendency to concentrate on certain routes that afford convenience, speed, and economy.

Factors That Affect Traffic.—The function of a highway is to provide a smooth, convenient route from the point of freight or passenger departure to the destination, and it must not be assumed that traffic will come to a road just because it happens to be a good road. Other factors enter the problem.

Certain roads acquire a name for being good roads and get the traffic apparently for no better reason than the Biblical allusion, "to him that hath shall be given". Likewise, the saying, "give a dog a bad name and it will stay with him", applies as well to highways and possibly explains why some roads lose traffic.

An apparently inconsequential matter that causes a road to miss a town may result in a loss of traffic that is out of all proportion to the advantage gained. The loss of traffic due to missing a town at a distance great enough to make access to the road difficult must be balanced by the traffic gained by the other, perhaps shorter, route.

Wellington states that "errors in the original laying out of a route, unlike errors in subsequent management, are mainly irremediable—a kind of fixed charge for folly forever". He also estimates that the loss of traffic* varies from 10 to 25% of the total traffic to be obtained by going directly through the center of the town for each mile a railroad passes at a distance from the center of the town. These figures, of course, apply only to railroads, but they give a guide as to a method of computing the effect of moving a highway away from a town.

The factors that cause variations in the location of a route are: Manufacturing towns that originate a large amount of traffic; special industries that must be served; the existence of competing routes; and special reasons that arise, and are peculiar to the existing conditions. An attempt to pass through the center of gravity of the population of two towns by going between them often leads to fallacious results unless more than two towns are considered.

Methods for reconnaissance surveys have not appeared in print so far as the writer has been able to determine. The method outlined herein is suggested as a procedure to be followed only as conditions justify.

The first step in a traffic reconnaissance survey is the collection of all available data in the form of published U. S. Geological Survey maps to be used in making notes of data and all reports as to motor vehicle registration in the countries to be traversed. Reports showing the location of industrial plants, tonnage of farm crops marked at various cities, census figures as to population of counties, and similar material, also will be useful. The various civic chambers of commerce can furnish valuable information.

Since it may be roughly estimated that about 85% of the motor vehicle travel will not go more than 50 miles on a single trip, and about 70% will not go more than 30 miles, it may safely be assumed in general that a margin of 50 miles, 25 miles on either side of the proposed line, will be sufficient to embrace the traffic that will be attracted to the road.

The next step is to make a careful study of the 50-mile strip of territory, estimating the tonnage to be hauled. This tonnage can easily be converted into the approximate number of motor-vehicle trips necessary to transport. To estimate pleasure car traffic the registration in the area may be multiplied by about 5 000 miles of annual use per year for each car, and this, in turn, may be changed to trips past a certain point with a suitable correction. Through traffic may be estimated from traffic counts at selected points, suitable corrections being made for seasonal variations and week-end concentrations.

The third step is to post men at selected points, determined by a study of maps and personal examination of the roads, and to inquire about traffic and such matters as the possible sale of filling stations or revenue from other

* "The Economical Theory of Railroad Location," p. 63.

similar sources. The men should count the traffic on selected days of the week covering 24 hours on each day.

From the traffic counts and the other data available, a rough approximation as to existing traffic can be obtained. This estimate can be corrected for attraction of traffic to the new road and the influence of competing roads. To determine future traffic, the potential traffic in the area based on tonnage hauled and registration predicted, as well as population increase, can be estimated as the maximum regional traffic. The determination of increase in through traffic is not so simple, but may be estimated from increased registration in the State, by considering that through traffic, in normal cases, is not more than 25% of the total traffic.

Future traffic may be estimated from the slope of the curves of traffic by years, especially the curve of motor-vehicle registration in this area. Predictions more than five years in advance are likely to be very misleading.

Evaluating Distance and Time.—The saving of shortened distance to transportation agencies is readily determined by a simple computation using the method of W. H. Connell, M. Am. Soc. C. E.* For example, with an operating cost of about 10 cents per mile and a traffic consisting of 85% light motor vehicles and 15% trucks, the cost for each 1 000 ft. over a paved road, or all-weather road, is about 2 cents for each vehicle. For 1 000 vehicles per day the cost would be about $20, or $7 300 per year. The saving per mile per year due to shortening the route is, therefore, approximately $38 700, or $774 000 over a period of 20 years, the life of many bond issues.

The value of a car-minute as determined for congested areas adjacent to New York City by the State of New Jersey in studies of highways was: For trucks, 2.3 cents; for buses, 2.1 cents; and for pleasure cars, 1.0 cents. In applying these figures to single grade crossings† it was found that on a highway carrying four lines of traffic averaging 20 000 000 vehicles per year there was a loss of 7 000 000 car-minutes, or $154 000 per year which, capitalized at 4%, equals $3 850 000, the sum that might reasonably be invested to eliminate this delay.

Location, Curves, and Grades.—The effect of location, curves, and grades on the first cost is direct and must be considered from the standpoint of the initial investment in the road—whether it is desired to build a cheap or expensive line.

Having determined from the consideration of the various traffic surveys, the traffic-source points through which the projected line must pass, the problem becomes one of finding the straightest and most economical line through these points. This line is located on the ground by field surveys of as many routes as seem feasible, using the methods familiar to highway engineers.

The elimination of grade crossings with railroads and intersecting highways is one of the largest items of expense and must be considered carefully because of the liability of a private corporation to damages for accidents occurring on its property. By observing care in locating the intersections of

* Report of Investigations of Paving and General Highway Conditions by the Commission Appointed by the National Paving Brick Manufacturers Association, p. 19.

† "Economic Theory of Highway Location," by F. Lavis, M. Am. Soc. C. E., *Roads and Streets*, October, 1927, p. 443.

highways so that they meet at different levels the cost of grade separation can be reduced. On level ground a treatment of the type shown in the sketch, Fig. 13, may be adopted. It is desirable to have as few intersecting highways as possible for safety considerations, but it is also desirable to have as many traffic sources as possible from the standpoint of local revenue. The private drive nuisance on public roads, which permits the entry of vehicles on the road at practically any point, is eliminated when the road is on a protected right of way with sharply defined places of entry and departure for traffic.

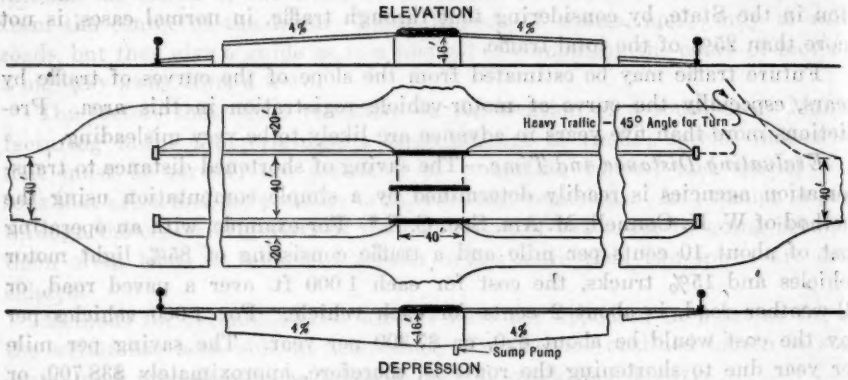

FIG. 13.—PLAN FOR HIGHWAY GRADE SEPARATION ON LEVEL GROUND BY EITHER ELEVATION OR DEPRESSION OF ONE ROADWAY.

The saving in time by the elimination of delays at crossings and other points of congestion of traffic will be an important factor in making the road more attractive to traffic, thereby increasing revenue. The capacity of the road is also increased by the time saving.

Signaling for traffic control while entering or leaving the road and en route should be reduced to a maximum necessary for safety, in order to avoid delays and annoyance for the traveler. The amount expended on signaling will vary with the number of danger points on the road.

Estimating Revenue.—Revenue to a toll road may ordinarily accrue from (1) use charges or tolls for various types of vehicles; (2) advertising space and concessions leased; and (3) rent of surplus land usually obtained in acquiring right of way.

Toll charges for various types of vehicles should be applied as simply as possible. In Italy, various categories based on the horse-power of the vehicles are in use; term or season tickets are issued, in addition to ordinary trip tickets.

The value of connecting routes as sources of both local and through traffic is self-evident. These routes also intersect other roads, in most cases, and one of the problems of the toll highway officials is to influence this traffic to use the toll roads because of the superior advantages offered. There also exists a competition with other roads that parallel the toll highway and here, again, a problem is evident.

From these and other factors it is apparent that the chief problem from the standpoint of revenue is the sale of transportation facilities which may

have sufficient advantages over other available facilities so that the traffic is willing to pay a premium to enjoy these advantages.

The sale of transportation, like the sale of anything else, is based on making the user of the facility want it, and the same effort and technique must be used as in the sale of any other commodity. In this sales effort the use of advertising plays an important part. Advertising campaigns based on the recommended procedure of specialists in such work are advisable and undoubtedly will have an important effect in increasing the travel on the road.

Some revenue will be derived from the leasing of billboard space on the right of way, but it has the disadvantage of obstructing the view of the landscape and makes the highway less attractive to the traveler. Concessions such as gasoline filling stations, telephones, refreshment stands, etc., are also a sure source of revenue.

The leasing of excess land owned by the company may be a source of revenue which, in some cases, may be used to offset the carrying charges of such land. It is inevitable that in acquiring right of way, whether by easement or purchase, there will be much of it not used for highway purposes. With the construction of the road this land will increase in value in most cases and may be sold later at a profit.

The ratio of local traffic (not more than 50-mile trip) as determined by traffic surveys in several parts of the United States, is from 10 to 25% of through traffic varying with the season, the section of the road, and the section of the country. It is quite evident, therefore, that the bulk of the revenue traffic will be local in nature and, at present, will be for the most part light pleasure automobiles. It is highly probable, however, that a decided increase in truck traffic will be seen within the next decade.

Construction Costs.—These costs, of course, depend much on the investment in roadway and protective works which the study of present and future traffic indicates will be profitable for the location selected. In determining the first cost of a pavement, the relative proportion of that cost necessary for maintenance must be considered as well as that of renewals at the end of its useful life.

Financial Study of a Toll Road.—For purposes of outlining a method of studying the working of a proposed toll-road project, consider a hypothetical case.

A toll highway is proposed to connect two cities of about 1 000 000 population each, that are about 80 miles apart. Between these cities are ten smaller cities varying in population from 5 000 to 50 000 which will be by-passed but tapped with short branch roads. There are ten entry points to the highway at the small cities and also ten other minor intersection points of industrial centers at which entry may also be made, making a total of twenty-two entry points, including the two terminals. Initial traffic is approximated at 4 000 miscellaneous vehicles per 24-hour day.

The investment in the project is estimated roughly, as follows:

80 miles of operating highway at $100 000	$8 000 000
Working capital stock issued as needed	2 000 000
Total	$10 000 000

The items that may be considered in the construction costs and the percentage of the total for each item for a road constructed at a cost of $100 000 per mile may be approximated, as follows:

	Estimated cost.	Percentage of total cost.
Right of way	$12 000	12
Grading and draining	15 000	15
Bridges	15 000	15
Crossing eliminations	25 000	25
Pavement	25 000	25
Miscellaneous protective works	5 000	5
Signaling and lights	2 000	2
Buildings	1 000	1
Total	$100 000	100

A statement of the annual income and expense of the report might be approximately, as follows:

Income.—	
Through traffic tolls, 1 000 vehicles per day at $1.00	$ 365 000
Local traffic tolls, 3 000 vehicles per day at 50 cents	550 000
Season tickets, 2 000 at $50.00	100 000
Advertising space leased	50 000
Concessions leased, filling stations, etc	50 000
Total	$1 115 000
Expense.—	
Fixed Charges:	
Interest on bonded debt of $8 000 000 at 6%	$ 480 000
Sinking fund to retire $8 000 000 in 50 years, amount annually at 5% interest	39 000
Operating expense, annual (see statement)	450 000
Proposed dividends on $2 000 000 stock at 6%	120 000
Surplus, annual	26 000
Total	$1 115 000

Operating Expense Statement.—	Estimated cost.		Percentage of total expense.
Maintenance of Pavement and Bridges:			
80 miles at $300	$24 000		
Miscellaneous at $50	4 000	$28 000	6
Renewal of Pavement and Bridges (20-Year Life):			
80 miles, $40 000 mile cost at 5% interest, per year		97 000	22
Maintenance of Protective Works:			
80 miles at $50		4 000	1
Renewal of Miscellaneous Works (10-Year Life):			
80 miles, $6 300 mile cost at 5% interest, per year		40 000	9
Collection of Tolls:			
30 men at $2 000 per year		60 000	13

Operating Expense Statement.—(Continued.)

		Estimated cost.	Percentage of total expense.
Policing (Motorcycle):			
4 men at $2 500 per year		$10 000	2
Executive and Clerical:			
Manager	$15 000		
Engineers	10 000		
Clerks	4 000		
Accountants	6 000		
Miscellaneous	$5 000	40 000	9
Damage claims and legal		15 000	3
Advertising and sales		20 000	5
Taxes		50 000	11
Insurance		8 000	2
Miscellaneous and contingent		78 000	17
Total		$450 000	100

G. F. SCHLESINGER,* M. AM. SOC. C. E. (by letter).†—The discussion of this paper has been of an informative and supplemental nature, as the subject-matter was not such as to call forth comment of a controversial character. Highway engineers and authorities are agreed that the information supplied by a comprehensive transport survey is essential to the proper planning of a program for future improvements.

Mr. Simpson calls attention‡ to the impracticability of securing as complete information in a city traffic survey as in the Ohio survey of rural highways, and questions the necessity of what might be termed non-essential data in solving problems arising from city traffic. As stated by the writer§ some of the information made available by the Ohio Transport Survey "might be classified as being outside the scope of practicality". However the data were obtained at very little additional expense and, without discussing their practical utility, it may be stated that they have been a valuable contribution to the general knowledge of the fundamental principles of highway traffic. It has occurred to the writer that a knowledge of the origin and destination of traffic would be very desirable in a city where conditions require the designation of a system of "one-way" streets.

It is somewhat surprising that, according to the Columbus survey, the proportion of traffic in the main city streets should agree so closely with that on the main Ohio rural highways (the State System). It would seem logical that the proportion on the main city streets, as compared to rural routes, would be less, due to the following causes:

(1) The greater availability of alternate secondary city streets;
(2) The better condition of the pavements on alternate secondary streets; and,
(3) The tendency of city traffic to avoid congested and traffic control (manual and automatic) conditions on the main thoroughfares.

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† Received by the Secretary May 2, 1929.

‡ *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2756.

§ *Loc. cit.*, September, 1928, Papers and Discussions, p. 2088.

It would follow that there must be other counteracting factors in the case of rural main highway traffic. There is probably a greater percentage of pleasure traffic outside the city, that does not follow the most direct routes. The fact that the maximum daily traffic occurs on Sunday in the country and that it is a minimum on that day in the city, according to Mr. Simpson, shows clearly the use many city motorists make of their cars over the week-end.

In his discussion, Mr. Van Duzer mentions* the importance of careful methods in forecasting future trends and also the securing of additional data that later might modify the general plan based on the original survey. There should be periodic "follow-up" surveys, which can be conducted on a much smaller scale. He also suggests the possible utilization of highway transport surveys in connection with the safe operation and control of traffic, with which, he states, future highway administration may be expected to concern itself. The operation of the State highways has been placed, to a considerable extent, in the hands of the State highway engineering executives, and properly so. Such duties as the erection of route markers and warning signs belong to them, and, in some cases, the highway police force is under their control. However, in the cities, the control of traffic is usually under the public safety, police, traffic, or other department without engineering administration and, in most cases, without engineering advice.

Mr. Thomas draws† an analogy between highway and railroad traffic. There is, however, a decided difference in respect to the composition of passenger traffic in the two types of transportation. The Ohio survey shows that 44.6% of the passenger-car traffic on the State Highway System measured in passenger-car miles is non-business or pleasure traffic, while railroad traffic is more predominantly of the business variety. Railroad passenger traffic, also, is not carried in small mobile units subject in routing to the almost instant direction of a relative large number of individual operators. For this reason it would not be accurate to assume that a "road properly improved will get all the traffic, much of which is now going by other routes".

Mr. Thomas quotes from the original paper to the effect that 14% of the traffic exceeded a 50-mile limit of trip. This figure applies to truck traffic and approximately 30% of all traffic exceeded 50 miles per trip.‡ However, Mr. Thomas is correct in his conclusion that highway traffic is primarily a function of local transportation and of the population of local areas tributary to the highway. The writer agrees that the formulation of a method for approximating the traffic on a given or projected highway route without the necessity of a field survey is within the range of possibility, using the data and principles established as a result of transport surveys already made.

Some of the principles established by the traffic studies in Ohio, and by other comprehensive surveys, have been applied by Mr. Thomas to the problem

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2759.
† *Loc. cit.*, p. 2763.
‡ *Loc. cit.*, September, 1928, Papers and Discussions, p. 2081.

of determining the probable commercial success of a proposed toll-road project. His analysis, suggested method of procedure, and application to a hypothetical case are admirable. Nor can the writer offer any valid objections to such private ventures in the transportation field, if they appeal to capital as profitable enterprises, assuming that the public interest receives the usual legal protection incident to quasi-public utilities. However, there is such decided opposition to toll roads in the United States that there is little probability of such enterprises being legally authorized until there is a great change in public sentiment. In the writer's opinion no legislative body, at the present time, could be persuaded to grant a toll-road company the right of eminent domain to condemn private property for the right of way required for such a project. While the gasoline tax is essentially a form of tolls, it has been termed a "painless tax" because of its method of collection. The motorist is not confronted with a barrier that will be closed to him until he has purchased a ticket. A projected toll road, as Mr. Thomas observes, would offer an excellent opportunity for the application of the principles of highway traffic flow in a purely economic study with social and political considerations eliminated. It is believed that in the future development of highway transportation in this country, as in the past, highway engineers will be required to deal with such social and political problems along with those of a technical nature.

That methods of approximating traffic are not only possible, but actually have been applied, is revealed in Professor Dougherty's description* of the surveys in connection with proposed toll-bridge projects in Tennessee. The statement† that "a 12-hour count taken in August, therefore, approximates the average 24-hour flow throughout the year", may give readers the impression—as it did the writer at first reading—that one 12-hour count during that month is all that is required for this approximation. The reference is to the average of daily 12-hour daylight counts taken during the month of August. The details of the analysis of probable traffic and consequent tolls are very logical and should be useful to those confronted with similar problems. The writer will inject the observation that the prevailing method of financing highway improvements by means of a tax on gasoline is essentially one of tolls.

Colonel Crosby calls‡ attention to the fact that for a century or more highway engineers have recognized the utility of traffic data as factors in the design of roads and the selection of types of pavements. The writer believes that a more important and valuable utilization is in the establishment of a State-wide or regional plan for a future program of highway development. He agrees with Colonel Crosby that there is a benefit to society in the improving of highways that is indefinable and incapable of measurement in terms of traffic accommodation, or any other material units.

* *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 1029.
† *Loc. cit.*, p. 1030.
‡ *Loc. cit.*, May, 1929, Papers and Discussions, p. 1265.

of determining the probable commercial success of a proposed toll road project. His analysis, suggested method of procedure, and application to a typical case are admirable. Nor can the writer offer any valid objections to such private ventures in the transportation field—if they appeal to capital as profitable enterprises, assuming that the public interest receives the usual legal protection incident to quasi-public utilities. However, there is a decided opposition to toll roads in the United States that there is little probability of such enterprises being legally authorized until there is a great change in public sentiment. In the writer's opinion no legislative body, at the present time, could be persuaded to grant a toll-road company the right of eminent domain to condemn private property for the right of way required for such a project. While the gasoline tax is essentially a form of tolls, it has been termed a "painless tax" because of its method of collection. The motorist is not confronted with a barrier that will be closed to him until he has purchased a ticket. A projected toll road, as Mr. Thomas observes, would offer an excellent opportunity for the application of the principles of highway traffic flow in a purely economic study with social and political considerations eliminated. It is believed that in the future development of highway transportation in this country, as in the past, highway engineers will be required to deal with such social and political problems along with those of a technical nature.

That methods of approximating traffic are not only possible, but actually have been applied, is revealed in Professor Dougherty's description* of the surveys in connection with proposed toll-bridge projects in Tennessee. The statement† that "a 12-hour count taken in August, therefore, approximates the average 24-hour flow throughout the year", may give readers the impression—as it did the writer at first reading—that one 12-hour count during that month is all that is required for this approximation. The reference is to the average of daily 12-hour daylight counts taken during the month of August. The details of the analysis of probable traffic and consequent tolls are very logical and should be useful to those confronted with similar problems. The writer will inject the observation that the prevailing method of financing highway improvements by means of a tax on gasoline is essentially one of tolls.

Colonel Crosby calls‡ attention to the fact that for a century or more highway engineers have recognized the utility of traffic data as factors in the design of roads and the selection of types of pavements. The writer believes that a more important and valuable utilization is in the establishment of a State-wide or regional plan for a future program of highway development. He agrees with Colonel Crosby that there is a benefit to society in the improving of highways that is indefinable and impossible of measurement in terms of traffic accommodation, or any other material units.

* Proceedings, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 1623.
† Loc. cit., p. 1630.
‡ Loc. cit., May, 1929, Papers and Discussions, p. 1263.

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PUMPED-STORAGE HYDRO-ELECTRIC PLANTS

Discussion*

BY A. H. MARKWART, M. AM. SOC. C. E.

A. H. MARKWART,† M. AM. SOC. C. E. (by letter).‡—Any electric generating device or method which promises to be economical in power production is of peculiar interest to the electrical utilities of the country. That attitude conforms with the enlightened policy which they have adopted, of service at the lowest cost consistent with continuous operation and sound expansion to meet the demands of present and future consumers. Naturally, an increase in the number of consumers and in quantity of power used will bring about a reduction of cost to the consumer. The electric utilities, not content with this, are always on the alert to discover ways to reduce cost, and, therefore, service rates, beyond that which would be possible by the ordinary process of mass production. To this end, much thought has been given lately to the idea of carrying peak loads on capacity units which will operate at low load factors. In this way the bottom and greater portion of system loads might be carried, in the interests of economy, on base-load plants, at load factors considerably higher than system load factors.

Methods of carrying peak loads have been studied in this country and abroad. Consideration has been given to devices for generating from a secondary source, such as electric storage batteries, steam accumulators in steam-electric generating systems, and hydraulic accumulators or pumped-storage hydro-electric plants, in both steam and hydro-electric generating systems. Consideration has also been given to devices for generating from a primary source, such as steam plants in hydro-electric generating systems, and Diesel engines. Because of their high cost, electric storage batteries have not been used extensively to carry peak loads. The development of large Diesel engines for central station use has not advanced, and it does not seem likely that the

* Discussion on the paper by W. W. K. Freeman, Assoc. M. Am. Soc. C. E., continued from May, 1929, *Proceedings*.

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‡ Received by the Secretary, February 23, 1929.

Diesel engine will be widely used to carry peak loads to improve the load factor of base-load plants. Pumped-storage plants may be suitable at times; the desiderata are (1) favorable natural conditions and (2) a low first cost.

In the light of efforts on the part of electric utilities toward increasing economy in power production, a discussion of pumped-storage plants as a means of solving peak-load problems in electric power stations, is of timely interest. The author has grouped them* into three types: (1) The pondage plant; (2) the storage plant; and (3), the part-head plant.

Pumped-storage installations are not recognized as standard practice in America; however, wherever they are physically possible, further study may indicate economic feasibility. In fact, pumped-storage plants must be viewed largely with respect to their economic aspects because, aside from the pumping equipment involved, they present no technical features not present in the usual hydro-electric plant.

The economics relate wholly to the cost of power at the load factor under which such plants must supply power as compared with the cost from other plants, either steam or other hydro-electric, at the same load factor. One may say that pumped-storage plants, particularly the pondage and storage type, must be used, in effect, to supply kilowatts of peak load instead of kilowatt-hours of energy. Consequently, in any given system, they must work in the peak of its load, which contains a negligible quantity of energy, instead of in the base of its load which contains the bulk of the required energy. The part-head type may be conditioned so as to supply both peak and energy.

With the exception of the pumping equipment, the part-head plant has the physical composition of an ordinary hydro-electric plant, and under equal conditions of water supply the operation and character of power are the same. In comparing the cost of power from this type with that from an ordinary hydro-electric plant, account must be taken, not only of the fixed charges on the capital investment and the usual operation, maintenance, and depreciation costs, but due allowance must be made for the cost, if any, of off-peak energy used in pumping. This has the effect of adding capital not necessary in ordinary hydro-electric projects. When such off-peak energy is supplied from steam plants, its cost would be substantially that of the fuel not otherwise burned; if supplied by run-of-stream hydro-electric plants at off-peak times, its cost would be nothing. Under such conditions of operation gross output would be used in the analysis of power cost; but, if such a plant supplied its own power for pumping, the cost of power would be determined solely by the usual hydro-electric annual costs, provided net output was considered.

There might be conditions of limited water supply and favorable topography that would make it feasible to install part-head plants, operating on the part-time storage or the flow-regulation principles, to carry peaks only. The one would be a part-head pondage plant, using natural flow and ordinary forebay regulation instead of re-pumped water, and the other a part-head storage type, if there were other plants down stream and if the natural flow of the side stream were sufficient.

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2459.

In order to view this subject in its essential aspect and to reduce the economic problem to its simplest terms, the writer proposes to confine his discussion to the pondage type or to any pumped-storage plant that operates on the short-time storage principle; that is, one that uses system off-peak energy, during low load periods, from other plants of the system, to generate on-peak energy during high load periods at the pumped-storage plant and at other plants down stream, if such exist.

The problem is similar to that which presents itself in the case of steam plants, operating at low load factors, which are used to comb the top of system loads that have load factors considerably less than 100% in order to permit hydro-electric plants to operate on the bottom of the load at load factors considerably greater than the system load factor. Often steam power is cheaper than water power at low load factors, and *vice versa* at high load factors. When this condition occurs, there will usually be a composite economy in power production if the system load is divided into two parts, and the bottom, or high load factor, is placed on water power and the top, or low load-factor, is placed on steam. This, of course, assumes that the capital cost of hydro-electric energy is not so low as to preclude the use of any steam and that fuel is not so cheap as to preclude the use of any water-power energy.

Because of the limitation on the quantity of water used, pumped-storage plants must operate on the top or low-load factor part of the load of a given system, where the quantity of energy to be supplied is negligible. Thus, they become capacity plants and their function is to improve the load factor of the base-load plants—hydro-electric or steam, or both as the case may be—to the end that the cost of system power will be a minimum. However, because of the remarkable over-all efficiency of modern steam plants, pumped-storage plants will be of no value as an aid to composite economy in power production unless the capital cost per kilowatt of installed capacity is relatively low. It is the low capital requirement and negligible fuel consumption that justify steam plants for the top or low-load factor load in a system using base-load hydro-electric power. The two kinds of generation facilities, that is, pumped-storage, and peak-load steam plants in a water power system, have a comparable function, and may be truly said to be in economic competition when the base-load plants are operated with water power.

In places where the topography and other natural conditions are favorable it is probable that the capital cost per kilowatt of deliverable peak, at the required, very low, load factor from a pumped-storage plant, will be enough less than that from an ordinary natural-flow hydro-electric plant, with forebay and afterbay regulation as may be required for a low load factor, to justify the installation of the former provided the off-peak power for pumping is available at a low cost. Consequently, a steam plant will usually be the alternative for a pumped-storage plant in any economic comparison; and the latter will be justified if a kilowatt of power at low annual load factors can be produced from it cheaper than from a steam plant. However, cost comparisons must be predicated on a common delivery point, since the steam plant has place utility.

In passing, the steam-accumulator electric plant might be mentioned as a possible alternative for a pumped-storage hydro-electric plant wherever the system base load is carried by steam plants. In such an arrangement the boilers of the base-load plant furnish steam for the accumulators during off-peak hours and the accumulator, in turn, supplies steam to its electric generator in order to furnish a low, load factor, on-peak energy, all in the manner in which base-load plants supply off-peak energy to elevate and storage water that operates pumped-storage plants over the peak. The steam accumulator is peculiarly adaptable in steam-electric generating systems; consequently, it is improbable that it would ever be the best alternative for a pumped-storage plant in which the base load is carried on hydro-electric plants. For the purpose of a generalized economic study and to make this discussion applicable to either hydro-electric or steam-electric generating systems, the ordinary steam plant, which is suitable in either system, has been chosen as the alternative for a pumped-storage plant.

When a pumped-storage plant is being considered for a system that requires additional capacity, it should be expected to carry only that part of the peak which would bring about a nice balance of system on-peak and off-peak energy, with due regard for the over-all inefficiency of the transmission, pumping and generating combination. The part of the peak of new load which such a plant should carry can be determined only by making an intensive study of the load curve of the system in which it will operate.

This will furnish the basic information necessary to determine the quantity of off-peak energy which is available in an existing system at various times, and the amount of on-peak energy to be supplied when different amounts of the peak are to be carried by pumped-storage plants. Likewise, it furnishes a means of measuring the annual cost of off-peak energy to be used for pumping in case fuel is used in its generation.

Two composite load curves similar to those used in other economic studies* are submitted in this connection. One is for a system load factor of 65% (Fig. 8 (*a*)), such as would obtain for regional systems in California, and the other for 45% (Fig. 8 (*b*)), such as would obtain for a metropolitan system, perhaps anywhere. For convenience, these curves were based on a system load of 400 000 kw. They are, in effect, composite daily load curves, 365 of which would contain, with a great degree of accuracy, all the kilowatt-hours that would be found under 365 actual daily load curves. They are necessarily inaccurate with respect to the 400 000-kw. peak because there would not be 365 peaks of such a magnitude. There is an appearance of error in the number of kilowatt-hours lying in the peak, because there would be few such peaks, perhaps only one, in the year. The degree of error would seem to be measured by the number of kilowatt-hours in about 364 such peaks lying in that part of the load above the average annual peak; but this seeming error does not exist in the composite curves, because the method used in constructing them took into account this very thing. Thus, in effect, the curves, while not being the exact shape of daily load curves, are such that they will

* "Power in California," by A. H. Markwart, M. Am. Soc. C. E., *Journal*, Franklin Inst., Vol. 204, 1927, p. 179.

reflect with a high degree of accuracy the kilowatt-hours that must be produced annually for a load of 400 000 kw. at 65% annual load factor for regional power systems in the one case, and of 45% for metropolitan systems. Similarly, the curves are sufficiently accurate to ascertain the kilowatt-hours in various divisions of the load to base load and peak load and the off-peak kilowatt-hours in the base load. In Fig. 8 (*a*) the dotted division line between the base-load plant and the peak-load plant is at 68.6% of the peak-load ordinate and, in Fig. 8 (*b*), it is at 48.5 per cent. The energy ratio, off-peak to on-peak, equals $1\frac{2}{3}$: 1, which is equivalent to a pumped storage over-all efficiency of 60% in both cases.

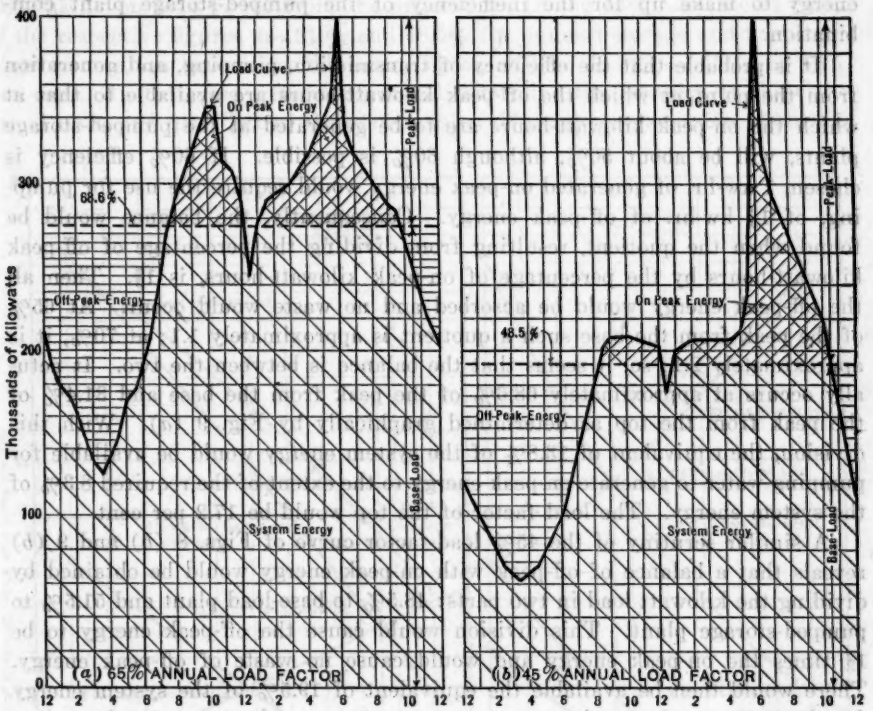

FIG. 8.—COMPOSITE DAILY LOAD CURVES.

To illustrate the use of the curves: If in Fig. 8 (*a*) 100% of the peak from the base, or 400 000 kw., is carried on steam, or on hydro-electric plants capable of operating at a load factor of 100%, these plants will supply 100% of the energy lying under the load line, or 2 277 600 000 kw-hr. at the system annual load factor; and by utilizing water otherwise going to waste, in the case of hydro-electric plants, or by burning additional fuel, in the case of steam plants, they would be capable also of supplying additional energy equal to 53.85% of the required system energy, or 1 226 400 000 kw-hr.*

Similarly, if 60% of the system peak from the base, or 240 000 kw., is to be carried on like base-load hydro-electric or steam plants, and the other 40%,

* *Journal*, Franklin Inst., Vol. 204, 1927, p. 179, table and diagram.

or 160 000 kw., is to be carried on pumped-storage hydro-electric plants, the latter would have to supply 16.37% of the system energy, or 373 000 000 kw-hr. The off-peak energy available for pumping purposes would be the equivalent of 8.77% of the system energy, or 200 000 000 kw-hr. Obviously, such division of load between the pumped-storage plants and base-load plants would not be feasible, because the on-peak energy is less than the off-peak energy. Fig 8 (*a*) shows an approximate equality, 11.90% against 11.80%, when 65% of the load is carried on base-load plants and 35% on pumped-storage plants. This is also an unbalanced condition because there is no margin of off-peak energy to make up for the inefficiency of the pumped-storage plant combination.

It is probable that the efficiency of transmission, pumping, and generation from the point at which the off-peak kilowatt-hours are available to that at which the on-peak kilowatt-hours are to be generated at the pumped-storage plants, will be about 50%, although 60% is possible. If 60% efficiency is chosen, 1 kw-hr. of generated on-peak energy would require the use for pumping, of $1\frac{2}{3}$ kw-hr. of off-peak energy. Consequently, the balance would be found when the quotient, resulting from dividing the percentage of off-peak kilowatt-hours by the percentage of on-peak kilowatt-hours, is $1\frac{2}{3}$. Then all the off-peak energy would be absorbed and no waste would occur. At 65% of the peak from the base such a quotient is approximately 1.1; at 70%, it is approximately 2.1; so, it seems that the balance is between the two. It actually occurs at approximately 68.6% of the peak from the base and 31.4% of the peak from the top as determined graphically by Fig. 9 (*a*). With this division, the equivalent of 13.8% of the system energy would be available for pumping water to generate on-peak energy to the extent of the required 8.3% of the system energy. The load factor of the top would be 17.2 per cent.

A similar scrutiny of the 45% load-factor curve of Figs. 8 (*b*) and 9 (*b*) reveals that a balance of off-peak with on-peak energy would be obtained by dividing the kilowatt load in two parts: 48.5% to base-load plant and 51.5% to pumped-storage plant. This division would cause the off-peak energy to be $1\frac{2}{3}$ times the on-peak energy and would cause no waste of off-peak energy. There would then be available the equivalent of 19.5% of the system energy for the purpose of pumping water to generate on-peak energy to the extent of the required 11.7% of the system energy. The load factor of the top would be 10.2 per cent.

It would appear, therefore, that if 65% and 45% represent the upper and lower limits, respectively, of load factors likely to be encountered in actual systems with triangular peaks, substantially all the off-peak kilowatt-hours of base-load plants can be balanced against on-peak kilowatt hours of pumped-storage hydro-electric plants. However, due allowance must be made for the over-all inefficiency of the pumping-storage combination when 31.4% of the peak for a 65% annual system load factor, and 51.5% of the peak for the 45% annual system load factor, are carried on pumped-storage plants. This would leave the remainder of the load in each case to be carried on water power or steam base-load plants.

The curves of Fig. 8 give other interesting information: For example, in the case of the 65% load-factor load, if the pumped-storage and base-load plant generation capacities are assumed to be 31.4 and 68.6%, respectively, of the load to be carried, motorization to the extent of approximately 37.6% (68.6% base load minus 31% minimum system load) of the load to be carried, or 20% more than generation, would be necessary. Thus, if all the off-peak energy is to be absorbed, the pumping load determines the capacity of the electrical unit when it is to have the double function of generator and motor. Likewise, the pumping load determines the capacity of the transmission line if one line can serve the dual purpose of bringing off-peak power to the pump and taking on-peak power from the generator. In the 45% load factor case, if the respective figures are 51.% and 48.5% for pumped-storage and base-load plant generation capacities, the motorization will be 48.5% — 15%, or 33.5 per cent. In this case, generation would determine the capacity of the electrical unit, and also that of the transmission, if single transmission obtained.

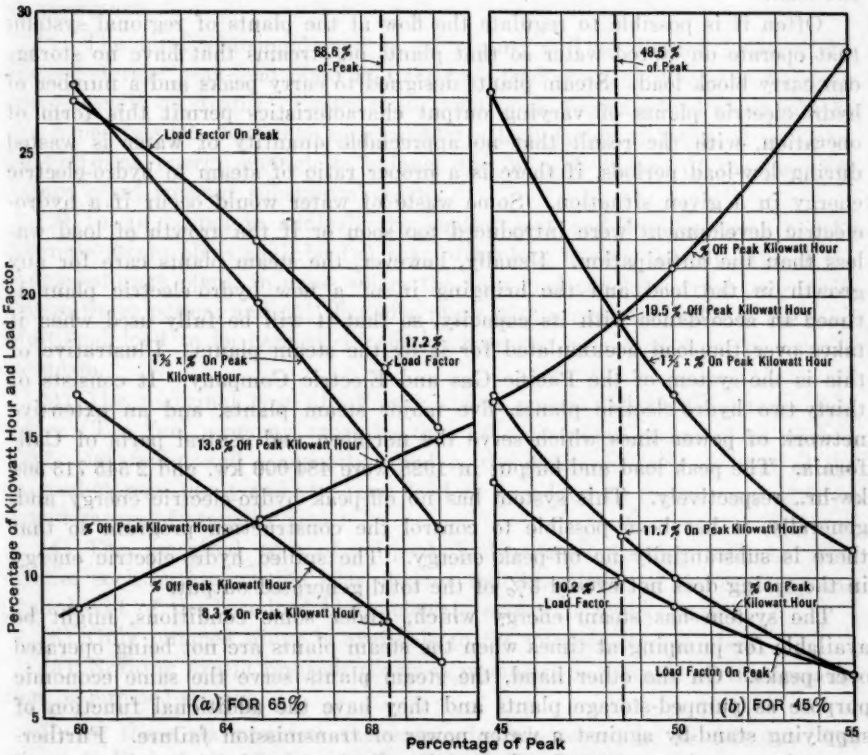

FIG. 9.—ANNUAL LOAD FACTOR SYSTEM.

Of course, the information relating to percentages of system peak available for pumping is of a general order only. It can be used as a guide to explore daily load curves for individual days to determine the relation of the off-peak and on-peak electrical capacities. However, the load of the minimum as well

as the maximum day, was included in making up Fig. 8. These composite curves are offered for what they are worth in suggesting a method to aid economic study only, rather than to determine the exact relation between motorization and generation for design purposes.

Another fact of some interest is revealed by reading the curves together. The quantity of off-peak energy that is available for pumping under conditions of practical balance between the two kinds of energy is reduced as system load factors rise. This is indicated by the 13.8% of the system energy that is available at the low-load period in the 65% load factor case as compared to the 19.5%, with a 45% load factor. From this it would appear that pumped-storage plants are more suited to metropolitan systems which have low system load factors, than to regional systems which enjoy high load factors. Furthermore, there is that other feature, which tends to bar their general use in steam-supported regional hydro-electric systems which usually enjoy high load factors, relating to the control of the water to suit the load factor of the load.

Often it is possible to regulate the flow at the plants of regional systems that operate on stored water so that plants on streams that have no storage can carry block load. Steam plants designed to carry peaks and a number of hydro-electric plants of varying output characteristics permit this form of operation, with the result that no appreciable quantity of water is wasted during low-load periods, if there is a proper ratio of steam to hydro-electric energy in a given situation. Some waste of water would occur if a hydro-electric development were introduced too soon or if the growth of load was less than the anticipation. Usually, however, the steam plants care for any growth in the load and the bringing in of a new hydro-electric plant is timed in accordance with its capacity, so that it will be fully used when it takes over the load accumulated for it by the steam plants. Illustrative of this is the system of the Pacific Gas and Electric Company. It consists of thirty-two hydro-electric plants, five major steam plants, and an extensive network of power lines which serve the northern and central parts of California. The peak load and output in 1928 were 484 000 kw. and 2 545 713 564 kw-hr., respectively. This system has no off-peak hydro-electric energy and, generally, it has been possible to control the construction program so that there is substantially no off-peak energy. The spilled hydro-electric energy in the spring does not exceed 3% of the total generated output.

The system has steam energy which, under some conditions, might be available for pumping at times when the steam plants are not being operated over peaks. On the other hand, the steam plants serve the same economic purpose as pumped-storage plants and they have the additional function of supplying stand-by against a water power or transmission failure. Furthermore, they supply energy to make up the water power deficiency in years of reduced run-off (when conditions are reversed and the steam plants are operated on the base of the load and the water power plants on the peak). Therefore, they can hardly be considered as means for supplying the energy necessary to the operation of pumped-storage plants. In fact, even if the base load of this system were carried on steam plants it is highly probable that

pumped-storage plants would be uneconomical because the water power sources that might be suited to such projects are at a considerable distance from the localities where steam plants are situated. However, it might be possible, in regional systems that have a metropolitan load, to utilize a pondage type of pumped-storage plant, providing a pump-storage site was available in close proximity to the steam plant in the metropolitan area, and that the economy in power production for the remainder of the regional system was not affected adversely.

The conditions that make pumped-storage hydro-electric plants economically feasible will vary in each locality; individual situations must be analyzed. The cost of peak power at the required low load factor from other available sources is the determining factor, and it is probable that usually steam power will be the alternative for pumped-storage power, with off-peak power from either steam or run-of-stream hydro-electric plants.

However, it is possible to state with approximate correctness, a general economic rule for pumped-storage hydro-electric plants under the following six premises:

(1) The total additional load to be carried will be divided between the pumped-storage peak-load plant and the base-load plant (either run-of-stream hydro-electric or steam). Under these conditions, the base-load plant will be of a capacity such that when operating at a load factor of 100%, it will produce off-peak energy enough to balance the on-peak energy of the pumped-storage plant without substantial waste.

(2) In an economic analysis the capital cost and fuel economy of the alternate steam plant shall be (*a*) that of a plant having the capacity of the pumped-storage plant, in cases where the base load can be carried on run-of-stream water power; and (*b*) that of a plant with a capacity equal to the total additional load to be carried, in cases where the base load must be carried on steam.

(3) The marginal cost of off-peak steam power will be for energy fuel only. (The fuel economy must be that of a base-load steam plant with a 100% load factor and a capacity equal to the difference between that of the pumped-storage plant and the total load to be added to the system, with no consideration of stand-by fuel.)

(4) The marginal cost of off-peak hydro-electric energy will be zero. (Under such an assumption it is obvious that knowledge of the capacity and capital cost of the base-load hydro-electric energy is unnecessary.)

(5) Only pumped water is available to the pumped-storage plant.

(6) Nothing except on-peak power is produced as a result of the installation of pumped-storage.

Under these six premises the economic rule would be as follows: The economic limit for capital expenditure for pumped-storage plants is the capitalized annual charges of steam plants operating under similar load conditions, as follows:

1.—In the event that the off-peak energy is supplied by run-of-stream hydro-electric plants the limit will be the annual cost of a kilowatt-year of equivalent steam power, capitalized at the annual percentage cost for hydro-electric plants.

2.—In the event that the off-peak energy is supplied by steam plants, the limit will be the annual cost of a kilowatt-year of equivalent steam power minus the annual cost of the fuel burned to supply a kilowatt of pumped-storage peak, capitalized at the annual percentage cost for hydro-electric plants.

It may be objected that these statements are mere truisms and are, therefore, of no significance, but truisms are significant if they cause the basic elements in what appears on its face to be a confused problem, to be universally recognized. Generalized statements of the elementary principles which attach to the problem under discussion are desirable in order to simplify its solution. The endeavor has been made to formulate such statements.

It is obvious that the allowable capital expenditure per unit of capacity for pumped-storage plants, for each price of fuel, will depend on the fuel economy and capital cost of the alternate steam plant, and also on the fuel economy under which the off-peak energy is generated if it is supplied from steam plants. The values of these elements, in turn, depend on the size of the plant. Consequently, economic comparisons must be made with like capacities and concomitant conditions. The capital cost per unit of capacity of steam plants and the fixed charges, that is, the interest, depreciation, taxes, operation, and maintenance cost, and the cost of stand-by fuel, or the annual charges except for energy fuel, decrease as the capacity increases; but the fixed charges, expressed as a percentage of the capital cost, will be substantially the same for all capacities of plant, varying only slightly for different unit prices of fuel. (See Table 3.)

In the case of a small pumped-storage plant a relatively high capital expenditure per kilowatt of capacity would be justified because its alternate steam plant would have a high capital cost and a relatively low fuel economy. On the other hand, the justifiable capital expenditure per unit of pumped-storage capacity for a large plant would be relatively low because the alternate steam plant in that case would have a low capital cost and a relatively high fuel economy.

It is probable that the total operating charges on pumped-storage plants would be close to those which would obtain for ordinary hydro-electric plants, and when expressed as a percentage of the capital cost, would be substantially the same for all capacities of plants.

An economic study under the rule will be made for a range of capacities from 5 000 to 100 000 kw., assuming that the annual fixed charges for steam plants are 15.5%, 15.75%, and 16% of the capital cost for $1.00, $1.50, and $2.00 oil, respectively, and that the annual fixed charge for pumped-storage plants is 10.25 per cent.

The annual fixed charge for a steam plant which is the alternate for pumped storage, operating in conjunction with base-load hydro-electric power, should be a little greater than that of the steam plant which is the alternate for pumped-storage operating in conjunction with a base-load steam plant. A little more stand-by fuel would be used in the first case, because it would be necessary to bring the alternate steam plant into operation each day from cold boilers in order to carry the load for a period of not more than two shifts, or about 16-hours. This kind of operation would be unnecessary in the

TABLE 3.—Relative Cost of Various Steam Plants.

Steam plant capacity, in kilowatts.	Capital investment per kilowatt, in dollars.	Interest, depreciation, and taxes at 12½, in dollars per kilowatt per year.	Operation and maintenance, in dollars per kilowatt per year.	Stand-by Fuel: Oil at 6 250 000 B. T. U. per Bbl.. Coal at 12 500 B. T. U. per Lb., or 4 to 1.							Total Cost per Kilowatt per Year of Steam Power. Not Including Cost of Energy Fuel and Annual Percentage Cost on Investment per Kilowatt.							
				Millions of British thermal units per kilowatt per year.	Barrels of oil per kilowatt per year.	Tons of coal per kilowatt per year.	Cost per kilowatt per year, in dollars.				$1.00 oil, in dollars.	$4.00 coal, per-centage.	$1.50 oil, in dollars.	$6.00 coal, per-centage.	$2.00 oil, in dollars.	$8.00 coal, per-centage.	
							$1.00 oil.	$4.00 coal.	$1.50 oil.	$6.00 coal.	$2.00 oil.	$8.00 coal.						
5 000	175	21.00	5.00	6.00	0.960	0.240	0.960		1.44		1.92		26.96	15.40	27.44	15.67	27.92	15.96
10 000	155	18.60	4.25	5.80	0.928	0.232	0.928		1.39		1.86		23.78	15.35	24.21	15.64	24.71	15.95
20 000	130	15.60	3.60	5.50	0.880	0.220	0.860		1.32		1.76		20.08	15.45	20.52	15.79	20.96	16.12
40 000	110	13.20	3.00	5.25	0.840	0.210	0.840		1.26		1.68		17.04	15.48	17.46	15.87	17.88	16.25
60 000	104	12.50	2.65	5.10	0.816	0.204	0.816		1.22		1.63		15.97	15.34	16.37	15.75	16.78	16.14
100 000	100	12.00	2.50	5.00	0.800	0.200	0.800		1.20		1.60		15.30	15.30	15.70	15.70	16.10	16.10

latter case, because the alternate steam plant would then be a part of the larger capacity that would be required to pull the total additional load, and it would be operated continuously instead of intermittently. However, this element is disregarded in the studies, because it is more or less academic; the effect, when considered, would not have been perceptible in the final result.

The fuel economies for the chosen capacities of alternate steam plants and base-load steam plants are intended to be those which would obtain with plants designed for moderate steam pressures and elevated temperatures ranging from 400 lb. with 700° total heat for the larger plants, to 250 lb. with 650° for the smaller ones. These conditions are generally used in steam plant practice and for that reason power resulting from such plants, rather than steam power resulting from the greater economies which would obtain with plants designed for elevated pressures and corresponding temperatures, is considered the alternate for power from a pumped-storage plant. However, in places where a pumped-storage plant of very large capacity is a possibility a steam plant with a 1 200 lb. capacity may be the alternate. In that case the study might properly be based on a fuel economy of about 11 500 B. t. u. per kw-hr., as compared with 13 000 B. t. u. (which is the highest value chosen for this study), except that no material change should be made in capital cost, because the investment per unit of capacity does not change as materially with changes in pressure and temperature, as it does with a change in size. The economic position of pumped-storage capacity would be slightly modified if a British thermal unit value per kilowatt-hour, based on the greatest advancement in the art of designing steam plants, were chosen in the studies. In most individual instances, this type of alternate steam plant would probably justify a smaller capital expenditure for pumped-storage plant than the moderate pressure type.

Assume that the pumped-storage plant or its alternate steam plant will generate at 17.2% load factor in the case of the 65% load factor, and at 10.2% load factor in the 45% case. This would seem to be necessary (see Fig. 8 (*a*) and Fig. 8 (*b*)) if the division of the load is to be such that it will bring about a balance between on-peak and off-peak kilowatt-hours after making allowance for losses. The energy requirements per kilowatt per year would be 1 507 and 893 kw-hr. for on-peak generation and 2 511 and 1 489 kw-hr. off-peak pumping, respectively, for the two cases. The quantity of energy fuel to meet this condition of operation for the various sizes of plants, used in this study, is given in Table 4.

The economical limits of capital expenditure for pumped-storage as determined by the rule are given in Fig. 10 (*a*) and Fig. 10 (*b*). The data for the curves were determined by using the cost of power from alternate steam plants with different prices for fuel. It is assumed that the off-peak pumping energy is supplied either by run-of-stream hydro-electric plants or base-load steam plants and that the annual fixed charge for the pumped storage is 10.25 per cent.

While this study is not precise and obviously cannot cover an individual situation, it sets with sufficient accuracy the upper limit of capital expenditure for pumped-storage plants of the capacities considered when they are to

operate on the top of 65% and 45% system loads having triangular peaks, if the cost of power therefrom is to be just equal to that of similar power from fuel plants. However, pumped-storage plants would not be economically attractive if the power from them was just equal in cost to alternate steam power; it would have to be considerably cheaper than steam power to offset the disadvantage which attaches to a plant that can be used only to "pull" peak loads. A steam plant can always furnish energy up to its capacity by burning sufficient fuel. Consequently, one would not build a pumped-storage plant in lieu of a steam plant unless the estimated capital cost of the pumped-storage plant was well below that which would bring an equality of cost with steam power as indicated by economic study; or unless, where base load is carried by both run-of-stream hydro-electric and steam plants, enough useful secondary power resulting from natural inflow to pumped-storage reservoir or from water pumped by surplus hydro-electric energy, was produced to reduce substantially the quantity of fuel otherwise burned in the base-load steam plants.

TABLE 4.—Quantity of Energy Fuel Required for Plants of Various Capacities.

Capacity of plant, in kilowatts.	British thermal unit required per kilowatt-hour.	Kilowatt-Hour Generated.		Fuel per Kilowatt Capacity.							
				With 17.2% Load Factor.				With 10.2% Load Factor.			
		Per barrel of oil.	Per ton of coal.	To generate 1 507 kw-hr. of on-peak energy per kilowatt of peak.		To generate 2 511 kw-hr. of off-peak energy per on-peak kilowatt.		To generate 893 kw-hr. of on-peak energy per kilowatt of peak.		To generate 1 489 kw-hr. of off-peak energy per on-peak kilowatt.	
				Oil, in barrels.	Coal, in tons.	Oil, in barrels.	Coal, in tons.	Oil, in barrels.	Coal, in tons.	Oil, in barrels.	Coal, in tons.
4 700	15 600	400	1 600	....	.....	....	.....	....	.....	3.72	0.931
5 000	15 520	403	1 612	3.74	0.935	....	.....	2.21	0.552	....	.....
9 400	14 800	421	1 684	....	.....	....	.....	....	.....	3.54	0.885
9 700	14 780	422	1 688	....	.....	....	.....	2.12	0.530	....	.....
10 000	14 740	423	1 692	3.56	0.890	....	.....	2.11	0.528	....	.....
10 900	14 640	425	1 700	....	.....	5.91	1.478	....	.....	....	.....
15 900	14 220	439	1 756	3.43	0.858	....	.....	....	.....	....	.....
18 800	14 040	445	1 780	....	.....	....	.....	....	.....	3.35	0.837
19 400	14 020	446	1 784	....	.....	....	.....	2.00	0.500	....	.....
20 000	13 990	447	1 788	3.37	0.843	....	.....	2.00	0.499	....	.....
21 800	13 910	449	1 796	....	.....	5.59	1.398	....	.....	....	.....
31 800	13 630	458	1 832	3.29	0.823	....	.....	....	.....	....	.....
37 700	13 530	462	1 848	....	.....	....	.....	....	.....	3.22	0.804
38 800	13 510	463	1 852	....	.....	....	.....	1.93	0.482	....	.....
40 000	13 500	464	1 856	3.26	0.815	....	.....	1.93	0.482	....	.....
43 700	13 440	466	1 864	....	.....	5.39	1.347	....	.....	....	.....
56 500	13 280	472	1 888	....	.....	....	.....	....	.....	3.15	0.789
60 000	13 230	473	1 892	3.18	0.795	....	.....	1.88	0.470	....	.....
63 700	13 200	474	1 896	3.18	0.795	....	.....	....	.....	....	.....
77 700	13 100	478	1 912	....	.....	....	.....	1.87	0.467	....	.....
87 400	13 040	479	1 916	....	.....	5.24	1.310	....	.....	....	.....
94 200	13 010	480	1 920	....	.....	....	.....	....	.....	3.10	0.776
100 000	13 000	481	1 924	3.13	0.782	5.22	1.305	1.86	0.465	3.10	0.776

Savings resulting from pumped-storage operation in any individual situation may be measured by the difference between the upper limit of capital

expenditure per kilowatt of pumped-storage capacity and the estimated cost per kilowatt of capacity in a given instance. For example: Consider a system that has an annual load factor of 65 per cent. Suppose that an increase of 60 000 kw. in capacity is contemplated; that a pumped storage site is available at which power can be developed at a capital cost of $200 per kw. of peak; that the base load will be carried on hydro-electric plants; and that the current price of coal is $6 per ton. Under such a set of conditions the 60 000 kw. of additional capacity would be divided into 18 840 kw. to pumped storage and 41 160 kw. to additional base-load water power. The former would operate at a load factor of 17.2% and the latter at 100 per cent. The upper limit of capital expenditure for off-peak energy from run-of-steam hydro-electric plants is $253 per kw., as given by the curve for the $6 fuel in Fig. 10 (*a*). Thus, the saving per kilowatt per year less than what steam peak power would cost at 17.2% load factor is 10.25% of the difference between $253 per kw. and $200 per kw., which is the estimated capital expenditure for the pumped storage, or $5.43 per kw. per year. If this saving, $102 300 annually, is sufficient to overcome the disadvantages inherent in pumped-storage plants, and not present in steam plants, the pumped-storage plant would be justified.

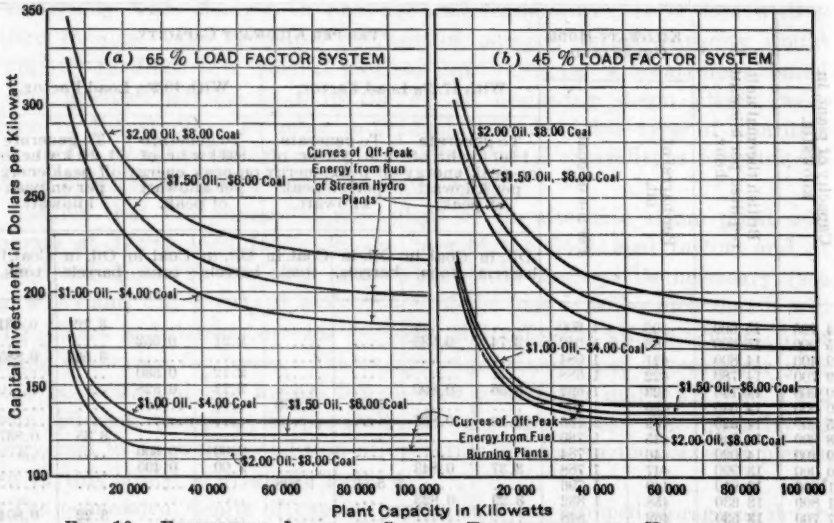

FIG. 10.—ECONOMICAL LIMIT OF CAPITAL EXPENDITURE PER KILOWATT OF INSTALLED CAPACITY.

Similarly, with a system load factor of 45%, off-peak power from base-load steam plants and an estimated cost of $200 per kw. for the pumped-storage plant, the additional capacity would be divided into 30 900 to pumped storage and 29 100 to base-load steam; the former would operate at a load factor of 10.2% and the latter at 100 per cent. The upper limit of capital expenditure as given by the $6 coal curve, off-peak energy for steam plants, is $140 per kw. (Fig. 10 (*b*)). In this case the excess cost of pumped storage over alternate steam power is 10.25% of the difference between $200 per kw., the

estimated capital cost, and $140, the limiting capital expenditure, or 6.15 per kw. per year ($190 000 per year). Obviously, this would be an uneconomical development.

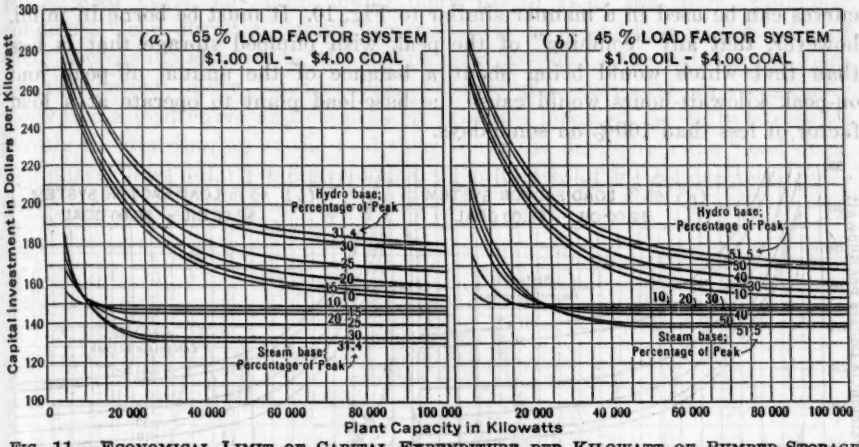
(a) 65 % LOAD FACTOR SYSTEM
$1.00 OIL - $4.00 COAL
(b) 45 % LOAD FACTOR SYSTEM
$1.00 OIL - $4.00 COAL
Capital Investment in Dollars per Kilowatt
Plant Capacity in Kilowatts
Hydro base; Percentage of Peak
Steam base; Percentage of Peak

FIG. 11.—ECONOMICAL LIMIT OF CAPITAL EXPENDITURE PER KILOWATT OF PUMPED-STORAGE HYDRO-ELECTRIC PLANTS FOR VARIOUS CAPACITIES AND DIFFERENT PERCENTAGES OF PEAK.

Thus far, the discussion has dealt with the maximum amount of peak load that could be carried by pumped-storage plants in the two load-factor cases on an annual basis. However, smaller amounts might properly be carried if for one reason or another it was not feasible to bring about a

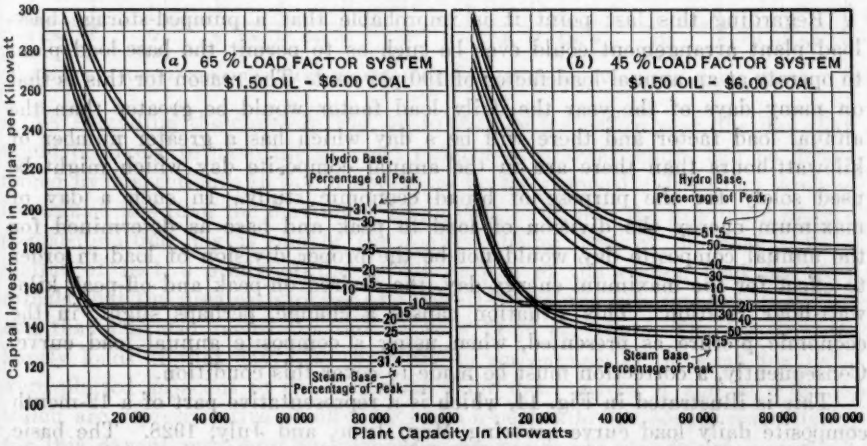
(a) 65 % LOAD FACTOR SYSTEM
$1.50 OIL - $6.00 COAL
(b) 45 % LOAD FACTOR SYSTEM
$1.50 OIL - $6.00 COAL
Capital Investment in Dollars per Kilowatt
Plant Capacity in Kilowatts
Hydro Base, Percentage of Peak
Steam Base — Percentage of Peak

FIG. 12.—ECONOMICAL LIMIT OF CAPITAL EXPENDITURE PER KILOWATT OF PUMPED-STORAGE HYDRO-ELECTRIC PLANTS FOR VARIOUS CAPACITIES AND DIFFERENT PERCENTAGES OF PEAK.

balance between the annual on-peak and off-peak energy of a given load to be added to a system. It is convenient to plot other curves, hydro-electric off-peak energy below and steam off-peak energy above the maximum curves, which would represent the limit of pumped-storage capital expenditure for

various percentages of the peak less than the maximum percentage at which annual off-peak and on-peak energy balance obtains. Tabulations are not included herein in this connection, but curves are given in Figs. 11, 12, and 13, for the two load-factor cases and for the three prices of fuel. These curves can be used in a manner similar to Fig. 10. It must be borne in mind, however, that any "combing" of the peak with pumped storage that is less than that which would bring about a balance of the annual off-peak and on-peak kilowatt-hours would cause the base-load plant to operate at a load factor of less than 100% on some days.

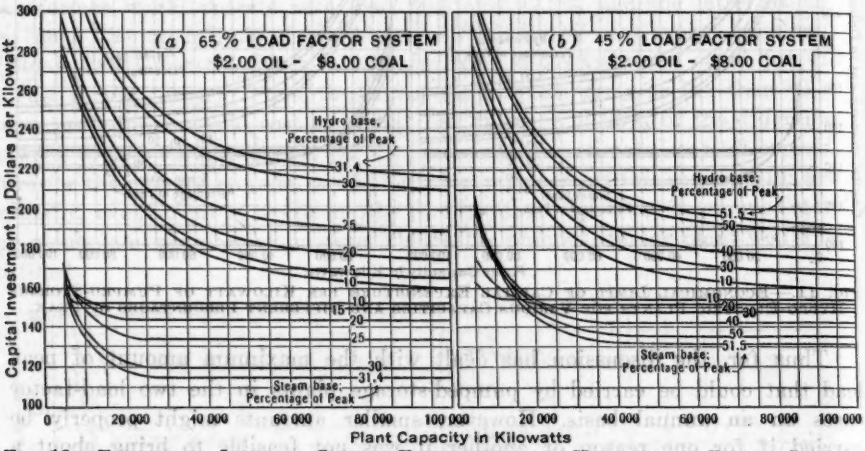

FIG. 13.—ECONOMICAL LIMIT OF CAPITAL EXPENDITURE PER KILOWATT OF PUMPED-STORAGE HYDRO-ELECTRIC PLANTS FOR VARIOUS CAPACITIES AND DIFFERENT PERCENTAGES OF PEAK.

Regarding this last point it is improbable that a pumped-storage base-load plant arrangement could ever be such as to permit the base-load plant to operate at an annual load factor of 100 per cent. The reason for this is that on many days of the year the daily load factor would be greater than the annual load factor and there will be a day which has a greater number of kilowatt-hours than there are in the annual composite day which might be used solely for the purpose of broad economic study. In such a day of maximum energy the division of load to peak and base as determined for the annual composite day would not be the proper division of load in order to effect for the maximum energy day, the desired on-peak and off-peak kilowatt-hour relation. This situation causes a change, perhaps slight, in the economic picture as presented, when using a composite annual load curve. Consequently, a correction must be made to cover this condition.

This is illustrated in Fig. 14, which is a representative part of a 12-month composite daily load curve covering May, June, and July, 1928. The basic data are the same as those used in Fig. 8. The January curve is the composite January day, the February curve the composite February day, and the other curves (through May, June, July, etc., to December) are corresponding composite monthly days.

The composite load curves illustrated in this study are made by plotting the half-hourly loads for the period considered (either a year or a month) on

a 24-hour base downward during the hours when the load is decreasing, and upward during the hours when the load is increasing. There were plotted 1 440 ordinates for the composite curve for a 30-day month (Fig. 14) and 17 520 ordinates for the composite curve for a year (Fig. 8 (*a*)). The composite load curve has been found more useful than the more commonly employed "load duration curve" constructed by re-assembling in the order of their decreasing magnitude, the ordinates representing the half-hourly loads for the period under consideration, because the composite curve actually looks like a load curve and closely approximates many of the daily curves; it reveals the distribution of the load throughout the day, which the "load duration curve" does not. For convenience, the ordinates of the composite monthly curves are plotted in percentage of the yearly peak.

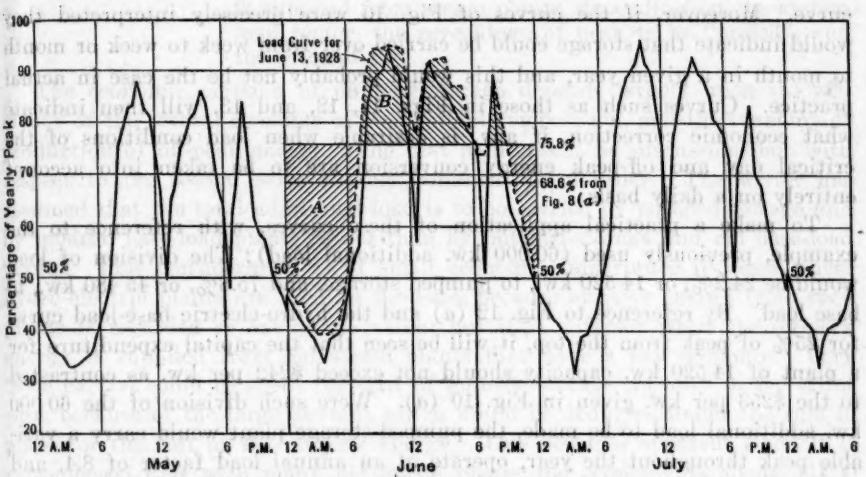

FIG. 14.—REPRESENTATIVE PART OF COMPOSITE DAILY LOAD CURVE FOR EACH MONTH.

By extending the curve in Fig. 14 to include the entire year, the day that would be likely to have the greatest number of kilowatt-hours can be determined by inspection. It would appear to be the composite June day because its peak is less than the maximum, being 95% of it, and the minimum of this composite June day is one of the highest, being 33% of the peak. As a matter of fact, a study of the actual records of the system load for 1928 reveals that the greatest number of kilowatt-hours was generated on June 13, and that the load factor for the day was 74.56 per cent. The actual half-hourly loads of that day, also expressed as percentages of the peak for 1928, were plotted adjacent to the composite June day. Areas *A* and *B*, Fig. 14, which are representative of off-peak and on-peak kilowatt-hours, respectively, for this day of maximum energy, were measured. It was found that the 1 to $1\frac{3}{4}$ relation was established when 75.8% of the annual peak was placed on the base for this day as contrasted with the 68.6% as determined by the study of the annual composite daily load curve, characteristic of the system in question (see Fig. 8). The on-peak energy above the 75.8% line, projected across the twelve composite monthly days, is equivalent to that above the

75.8% line on the annual composite curve in Fig. 8. The load factor and percentage of kilowatt-hours from peak will be 8.4% and 3.10%, respectively. These relations are to be determined by the use of a diagram similar to that given on Fig. 9.

The additional curves in Figs. 11, 12, and 13, are particularly useful in that they afford means of determining the economic situation respecting pumped storage if a study of the load curve of an individual day indicates that more load must be carried that day on the base-load plant than is indicated by the study of the annual composite daily load curve. The necessity for considering the maximum energy day becomes apparent when it is remembered that the curves of Fig. 10 are based on a composite daily load curve for the entire year, no actual day of which will be exactly like this composite curve. Moreover, if the curves of Fig. 10 were precisely interpreted they would indicate that storage could be carried over from week to week or month to month in a given year, and this would probably not be the case in actual practice. Curves such as those in Figs. 11, 12, and 13, will then indicate what economic correction, if any, is desirable when load conditions of the critical day and off-peak energy conversion are to be taken into account entirely on a daily basis.

To make a practical application of these curves, with reference to the example, previously used (60 000 kw. additional load): The division of load would be 24.2%, or 14 520 kw., to pumped storage, and 75.8%, or 45 480 kw., to base load. By reference to Fig. 12 (*a*) and the hydro-electric base-load curve for 25% of peak from the top, it will be seen that the capital expenditure for a plant of 14 520 kw. capacity should not exceed $242 per kw. as contrasted to the $253 per kw. given in Fig. 10 (*a*). Were such division of the 60 000 kw. additional load to be made, the pumped-storage plant would carry a variable peak throughout the year, operate at an annual load factor of 8.4, and supply 3.10% of the system kilowatt-hours. This detail is given to indicate a procedure which might be used to determine the capital limitations of pumped storage with greater refinement, if the maximum daily load factor is substantially more than that of the annual composite load curve which might have been used in an over-all economic study. The example chosen indicates that the limit of capital expenditure for pumped storage should be reduced about 5% less than that determined without giving consideration to the conditions which obtained for the maximum-energy and load-factor day and those which would prevail under daily instead of period carry-over storage operations.

The curves in Figs. 11, 12, and 13, reveal a number of interesting facts relating to the economics of such peak-load plants:

1.—Hydro-electric off-peak power warrants a larger capital expenditure than steam off-peak power.

2.—The limit of capital expenditure decreases:

(*a*) With increase in fuel price when off-peak power is supplied from steam plants; and

(*b*) With decrease in fuel price when off-peak power is supplied from run-of-stream hydro-electric plants.

3.—As the load factor decreases, the limit of capital expenditure:

(*a*) Increases when off-peak power is supplied from steam plants; and

(*b*) Decreases when off-peak power is supplied from run-of-stream hydro-electric plants.

4.—The limit of capital expenditure decreases with increasing plant capacity:

(*a*) With no tendency to become a constant for capacities under 100 000 kw. when off-peak power is supplied from run-of-stream hydro-electric plants, irrespective of the system load factor, the percentage of peak to be pulled, and the price of fuel;

(*b*) With a tendency to become a constant for capacities of more than from 10 000 to 55 000 kw. when off-peak power is supplied from steam plants, depending on the system load factor, the percentage of peak to be "pulled", and the price of fuel.

The economic position of pumped storage depends largely upon the efficiency of the several elements of its physical composition, and the economy of production of off-peak energy. The first point needs no discussion, but with respect to the second some further comment is in order. The writer has assumed that the total additional load is to be carried by pumped-storage and by modern base-load plants rather than by pumped-storage and old base-load plants. This consideration is of small economic importance in cases where hydro-electric plants are used on the base, because off-peak energy from such plants is regarded as having no value; but it is a factor when the off-peak energy is supplied from old steam plants that have a low economy. Very little capital could be spent per unit of pumped-storage capacity in the latter case, because of the large amount which would be deducted under the rule to allow for the cost of the off-peak energy. This becomes apparent when it is remembered that with many old steam plants the economy is about 30 000 B.t.u. per kw-hr., and because only a few of the old installations have been replaced with modern plants with high efficiencies, the average for all plants of the country is about 20 000 B.t.u.

It is probable that the capitalized value of off-peak energy based upon such economies would reduce the capital which could be spent per unit of pumped-storage capacity, so as to make impossible the building of such plants. However, if physical conditions were so fortunate as to make it possible to build a pumped-storage plant within a capital sum per unit of capacity that was determined on the basis of using low-economy off-peak energy, the reduction in cost per kilowatt per year (when the off-peak energy was finally supplied on a high-economy basis) would equal the difference between the low and high-economy fuel costs of off-peak energy per kilowatt per year. However, this is not likely to occur when very low-economy off-peak energy is available. For example, a fuel economy of 30 000 B.t.u. per kw-hr., with about $4 coal, would cause the cost for off-peak energy to be about $12 and $7 per kw. per year, respectively, for the 65% and 45% load-factor loads, in turn requiring pumped storage, in order to compete with alternate steam, to be procurable at from $60 to $100 per kw. It seems improbable that pumped storage could be built

at a cost of $100 per kw., or less, when that must include all transmission, pumping, generation, and storage facilities.

Low-economy off-peak power will occasionally be used with pumped storage, despite the probability that most pumped storage is likely to be barred if off-peak energy is regularly to be supplied by old steam plants. For example, it might be quite inconvenient to build a complement of base-load steam at the time when it is most necessary to build pumped storage, which estimates showed could be physically accomplished, provided the capital allowance was established on the basis of supplying off-peak energy from a high-economy steam plant. In such a situation the pumped-storage plant might properly be constructed, and for a time operated with off-peak power from old steam plants, with no disadvantage, save that the cost of energy from the pumped-storage plant would be high during the interim pending the completion of the new steam plant. The excess in cost per kilowatt per year would equal the difference between the low and high-economy fuel costs per kilowatt-year.

The best position for pumped storage with steam-electric power is that in which the base load is carried on new high-economy steam plants. This element, that is, the efficiency of operation of the base-load plants, is not important when the base load is carried on run-of-stream hydro-electric energy, because then, since off-peak energy is considered valueless, pumped storage would have its greatest opportunity and much of it could be built which otherwise might not be feasible where the base load is carried on steam.

Owing to the fluctuating price of fuel, judgment must be exercised in selecting the fuel cost to be used in an economic study. To be on the safe side it would be the part of wisdom, when off-peak power is to come from steam plants, to choose a long-time price of fuel of more than that which might obtain at the time a pumped-storage plant was being considered, if then the price was low; and when off-peak energy is to come from run-of-stream hydro-electric plants, one that was less than that prevailing at the time, if then the fuel price was high. This reasoning follows from the fact that a lowering of the economic limit of capital expenditure for pumped storage, in cases where off-peak energy is supplied from steam plants, comes with an increase in the price of fuel, but if the off-peak energy is supplied from run-of-stream plants, it comes with a decrease in the price.

This generalized study of the economics of pumped storage is methodical only and is intended to be suggestive rather than dogmatic. There are doubtless other ways to portray the situation, particularly if the cost of pumped storage is known. The endeavor herein has been to set up something without a knowledge of the pumped-storage capital costs. Other cost assumptions will show different generalized results. However, it is believed that the curves of capital limit indicate orders of capital cost for pumped-storage plants that cannot be materially exceeded if the resulting cost of power is to be no greater than the current value of power as established by modern, fairly high-economy steam plants. It should not be inferred that this analysis will obviate the necessity for intimate study in individual situations, but it will be helpful in identifying the pumped-storage projects which, in connection with appropriate base-load plants, are worthy of intensive study to determine

the cost of mingled energy from a complement of such plants. The method may be used for the purpose of generalizing with other load factors and under the same or other assumed data.

It is apparent to those in the power industry that one must be critically discerning when a determination is to be made in the matter of future power supply. What is desired is the lowest-cost electricity, irrespective of whether it comes from hydro-electric or fuel-burning plants, or from a combination of both. All recognize that, with annual system load factors which are always less than 100%, there will be an annual economy if the narrow peaks of the system load can be carried by plants which can be built at a very low capital cost per unit of capacity. This applies to almost any kind of a peaking plant.

The future course of power development necessary to carry total loads is, in general, not fully established. At present, steam power appears to be more or less in the ascendant because fuel is cheap and fuel economies are high. On the other hand, the abundance of cheap capital and the economic advantages which accrue by the use of high transmission voltages, causes hydro-electric energy to remain a source of power not to be overlooked, particularly when it is remembered that the price of fuel may be higher in the case of steam power, and sufficient experience has not been accumulated to determine with certainty the cost of operation and maintenance of modern steam plants; whereas the fixed charges which make up the major costs of hydro-electric power remain fairly stationary over a relatively long period of years. Thus, the time to build hydro-electric plants is when capital is cheap and it is sound business, if capital is readily obtainable, to use hydro-electric power when its cost is equal to or less than that of steam power. However, the economic picture in the hydro-steam scene is a moving rather than a fixed one; consequently, the engineer must be cognizant of economic conditions generally, in order that he may shift from one to the other or to both of these major power sources in delving for reduced costs. Because of the rapidly changing conditions, sound economic analysis is more important in the field of electric generation than it has been heretofore. In conclusion, be it said to the credit of the civil engineer that he is getting a broad grasp of the electric industry and is being increasingly recognized by it as a factor in the field of economic energy production.

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PAPERS AND DISCUSSIONS

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COMMENTS ON THE DESIGN OF SLUDGE DIGESTION TANKS

Discussion*

BY KENNETH ALLEN, M. AM. SOC. C. E.

KENNETH ALLEN,† M. AM. SOC. C. E. (by letter).‡—In 1897, the Englishman, W. D. Scott-Moncrieff, carried out a most interesting experiment§ in passing septic tank effluent through a series of nine trays placed one above another, each containing 6 in. of coke. No deposit had accumulated in 3 months of operation while, in the 8 or 10 min. required for passage through the series, a high degree of purification had taken place. This was attributed in part to the large proportion of liquifying bacteria in the septic sewage which converted much of the organic solids to a condition easily attacked by nitrifying bacteria. The development of these bacteria, in turn, was promoted by the oxygen absorbed by the sewage in dropping from one tray to the next; but the rapidity of purification was also attributed to the selective development, in each tray, of the particular bacteria specially adapted to their environment. Consequently, their activities were carried on under the most favorable conditions, just as division of labor in a well-organized industrial plant brings about the highest productive efficiency. Basing conclusions on "oxygen consumed" tests, Dr. Samuel Rideal found an average of 93%, and on the oxidation of nitrogen, he found that 91.6% purification was brought about thus in approximately 8 min. In commenting on these findings he stated that‖ "the nitrate has developed with extraordinary rapidity and to an extent that exceeds any other process."

The advantages of segregating the several steps of sewage treatment have been recently brought forward by Wynkoop Kiersted, M. Am. Soc. C. E.,¶ in

* Discussion of the paper by Richard H. Gould, M. Am. Soc. C. E., continued from April, 1929, *Proceedings*.

† San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

‡ Received by the Secretary, April 24, 1929.

§ Described in the 17th Annual Report of the Provincial Board of Health, Ontario, and in *Engineering Record*, Vol. XXXVIII, p. 561.

‖ *Journal*, Sanitary Inst., Vol. XIX, Pt. IV.

¶ *Municipal News and Water Works*, March, 1929.

a plea for so complete a separation of suspended solids by preliminary treatment that oxidation may be rendered easier and more rapid and secondary sedimentation avoided altogether. This is advocated as an ideal for which to strive.

In the activated sludge process, Martin points out the irrationality of utilizing compressed air simultaneously for agitation and aeration. The supply of air required for agitation and that for the supply of oxygen to the sewage are unrelated; and in the attempt to secure adequate treatment an excess for either one or the other of these purposes will ordinarily be used. Martin states,* "this difficulty would be avoided if the stirring and the aeration were effected independently." In connection with septic and Imhoff tanks, Dr. Willem Rudolfs has stated that† sedimentation and digestion, inasmuch as they are two distinctly separate processes, cannot be combined for the best possible results because they are unlike in principle. Mr. H. C. H. Shenton has expressed the same opinion.‡

These examples illustrate the trend toward a separation of the functions involved in sewage treatment in general, in order that each may be carried out under the most favorable conditions.

In sludge digestion as ordinarily practiced, there is a tank influent of fresh sludge with, perhaps, a percentage of ripe sludge for seeding. After entering the tank, which is usually built as a vertical cylinder, this material is intimately mixed with the other contents by circulation or by mechanical stirring. It has been found that this mixing process promotes bacterial activity with the formation and release of gas and, at the same time, retards the formation of scum. To make room for the inflowing sludge an equal volume of the contents of the tank is drawn off as supernatant or as digested sludge.

If the process is properly controlled for a sufficient period, the sludge thus withdrawn is dark in color, drainable, and inodorous; but as it comprises a part of the sludge that has entered the tank each day for, say, 40 days, representing the length of storage, it is evident that it cannot all be completely digested. The question arises as to whether or not a gradual progression of the sludge in a long horizontal tank of relatively small cross-section—provided with stirring apparatus to ensure a thorough mixing in that particular cross-section at a number of points—would not produce an equal result in less time or a more thoroughly digested product in the same time. It is probable that, with the varying condition of the sludge during the period between its entrance and discharge from the tank, there would be a corresponding variation in the specific type of bacteria, requiring different pH control and temperature for optimum results. In a tank of this kind it would seem fairly reasonable to approximate these requirements at perhaps three or four points.

With the exception of control, the Merchantville-Pensauken tank, converted about 1924 from a septic tank to a digestion tank, complies in prin-

* "The Activated Sludge Process," Chapter XXI, p. 249.

† *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 1065.

‡ "Common Defects at English Sewage Disposal Works."

ciple with the writer's suggestion.* As reconstructed, this tank is in the form of a baffled channel 6 ft. wide, 10 ft. deep, and 240 ft. long, giving a capacity of about 2 cu. ft. per capita for 11 000 persons. The sludge flows very slowly through this channel until it is drawn at the farther end, some time previous to which it has become black and thoroughly digested. The sewage flow averages about 1 000 000 gal. per day. In March, 1927, samples drawn at ten different points between the inlet and outlet ends showed that† "all samples after Station 5 were well digested". In a test made in February, 1927, the pH rose from 6.0 in the influent to 7.4 at Station 6 and 7.6 at the effluent. Between the sixth and eighth stations, gas evolution was very vigorous and the final product, after 9 months retention, was black, granular, tarry in odor and had a 75% reduction of biological oxygen demand. In a week or ten days of warm dry weather it will shrink from a depth of 13 in. to 4 in. on the drying beds and then can be heaped up and burned to a fine ash without additional fuel. According to Maj. M. J. Blew, Research Engineer, North East Sewage Treatment Plant, Philadelphia, Pa., the time of retention can be reduced about 20% by returning the tank supernatant back to the influent of the sedimentation tank, and it can be further reduced to 4 or 5 months by seeding with digested sludge, while by temperature control it can probably be cut to 6 weeks, or 2 months. The satisfactory results secured without pH control or artificial heating would undoubtedly be exceeded considerably with the adoption of such measures. In applying the principle of progressive digestion, two general types of construction are possible. By the first, the tank would consist of a long conduit slightly below the hydraulic grade, but with a gentle upward gradient to enable the released gases to collect in a dome at the outlet end. Baffles at the inlet would distribute the incoming sludge, seeded and limed, if necessary, and provision for pH control and for the removal of deposits, if formed on the rounded invert, by flushing or mechanical means, would be made at several points between the two ends of the tank.

By the second arrangement digestion would take place by steps in several separate tanks operated in series, each under independent control, the units of which could be of the "Boonton" type, as shown in Fig. 2,‡ or of the usual Dorr type.

A preliminary digestion in Imhoff tanks followed by completion of the process in a relatively small separate tank, as advocated for large plants by Karl Imhoff,§ M. Am. Soc. C. E., is in accordance with this idea. The sludge chamber in the Imhoff tank would be of smaller size and the partly digested contents pumped in a condensed form to the separate tank. Acid fermentation is avoided and optimum temperatures can be provided, all with minimum sludge capacities.

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* *Public Works*, June, 1924, p. 184, July, 1924, p. 213, November, 1927, p. 409, and December, 1927, p. 468.

† *Loc. cit.*, December, 1927, "Merchantville-Pensauken Sewage Treatment Plant," p. 470.

‡ *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2663.

§ "The Arithmetic of Sewage Treatment Works," by Karl Imhoff and Gordon M. Fair, p. 37.

Prüss has elaborated the same idea.* He computes the profit (after paying capital and operating charges) in liters of sludge gas per day per capita when selling at 6 pfennigs per cu. m. With single-stage digestion at either 12° or 25° cent., the surplus or profit is 0.2 liters and 3.0 liters with two-stage digestion when heating the second unit, only, to 25° cent.; that is, if the surplus gas were sold—as is customary in Germany—the net profit with two-stage digestion would be fifteen times that with a single tank.

Whether the cost of providing more than two stages to the process would be justified is not known, but if there is an economic advantage in adding an auxiliary tank to the sludge chamber of an Imhoff tank there would appear to be a still greater economy in providing optimum conditions for all stages in a single tank near the ground surface.

* "Fortschritte in der Ausfaulung von Abwasserschlamm," von Dr. Ing. Max Prüss, 1928.

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PAPERS AND DISCUSSIONS

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ELASTIC EQUILIBRIUM IN THE THEORY OF STRUCTURES

Discussion*

BY MESSRS. JOHN I. PARCEL AND S. TIMOSHENKO.

JOHN I. PARCEL,† M. AM. SOC. C. E. (by letter).‡—This excellent paper is a timely contribution to the theory of structures. It is very true, as the author notes,§ that the buckling of beams has received far less attention than the buckling of columns. There are doubtless good reasons why the latter should have taken a more prominent place in the literature of the subject, but it is none the less true that the "elastic equilibrium" of beams has not been given the place its importance deserves, at least in American technical literature.

The author states§ that, so far as he knows, this and two other brief papers constitute the only analytical treatment of the problem. Without any intention of minimizing the importance of Mr. Richmond's independent and original analysis, it seems pertinent to call attention to the very considerable literature on the subject that has been in existence for some time.

The first treatment of the problem appears to be due to Professor L. Prandtl, of the University of Göttingen. His original paper‖ covers very fully the basic theory of the "tipping" of long, thin beams in flexure and includes a detailed treatment of what the author designates as "*J*-beams".

The first study of flanged beams, I-sections in particular, appears to be due to Professor S. Timoshenko.¶ His treatment was on essentially broader lines than that devoted to "*F*-beams" in the paper, covering quite fully the

* Discussion of the paper by H. S. Richmond, Esq., continued from May, 1929, *Proceedings*.

† Cons. Engr. (Sverdrup & Parcel), St. Louis, Mo.

‡ Received by the Secretary, April 1, 1929.

§ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 5.

‖ "Kipperscheinungen," (Munich Dissertation), 1899. A very clear and excellent résumé of Prandtl's theory will be found in the treatise by H. Lorenz, "Technische Elastizitätslehre" (1913), pp. 353–365.

¶ First presented (in Russian) in a *Bulletin* of the Technical Inst. of St. Petersburg, 1906.

case referred to by the author* as the "flanged beam complicated by the existence of the J-property". In later papers,† Professor Timoshenko has developed his theory in some detail for both simple and cantilever beams of I-section under a variety of loadings.

The interesting fact that a channel section, in order to develop a pure bending when loaded in a plane parallel to the web, must have the load placed a certain distance back from the web (see Fig. 5‡), appears to have been first observed by Professor C. von Bach. The analysis of the flexure and torsion of channels has been the subject of many papers appearing in the German technical press since about 1920.§

Although the author presents some very interesting and useful tables and graphs, the prime object of his paper seems to be the presentation of a fundamental method for the rational analysis of the problem of the stability of a narrow beam subjected to flexure. In view of this fact, the presentation of a somewhat different method of approach leading to the same results may be of some interest.

The writer assumes Equations (4) to (7), inclusive,‖ as the basic formulas, except that the right side of Equations (4), (6), and (7) is assumed to be negative. The author notes that in a flanged beam fixed at one end and carrying a concentrated load at the other, the principal torsional resistance is furnished by a lateral bending moment in the flanges, $\pm M_3$. Assuming that all distortions are small enough to justify the application of the theory of elasticity, it is possible to write, with sufficient exactness:

$$\eta = \frac{b}{2}\,\alpha \qquad (60)\P$$

in which, η = the linear displacement of the center line of the flanges in a direction normal to the plane of the web, due to the twisting effect alone; and from the ordinary theory of bending,

$$\pm M_3 = E I_f \frac{d^2 \eta}{d x^2} \qquad (61)$$

if I_f = moment of inertia of one flange in the plane of bending. Furthermore,

$$\pm \frac{d M_3}{d x} = E I_f \frac{d^3 \eta}{d x^3} = \frac{b}{2} E I_f \frac{d^3 \alpha}{d x^3} = V \qquad (62)$$

The pair of oppositely directed shears in the top and bottom flanges, due to the lateral bending moment, M_3, effectively constitute a couple:

$$V b = b \frac{d M_3}{d x} = \frac{b^2}{2} E I_f \frac{d^3 \alpha}{d x^3} = K \frac{d^3 \alpha}{d x^3} \qquad (63)$$

if $K = \frac{b^2}{2} E I_f$.

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 15.

† "Einige Stabilitäts Probleme der Elastizitätstheorie," *Zeitschrift f. Math. u. Phys.*, 1910, Vol. 58, pp. 337–385; and "Sur la Stabilité des Systèmes Elastiques," *Annales des Ponts et Chaussées*, Vol. III, pp. 496–566; Vol. IV, pp. 73–132; and Vol. V, pp. 372–412.

‡ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 15.

§ See paper by Professor L. Föppl, *Der Bauingenieur*, Heft. 12, 1925, p. 455, *et seq.*, in which a considerable bibliography is given.

‖ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 10.

¶ This is essentially the same as the author's equations, *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, top of p. 13.

If, in addition to the resistance to twist furnished by the lateral bending of the flanges, the beam also develops a purely torsional resistance, it follows that, from Equations (4), (5), and (63):

$$Q = P\left(y - x\frac{d\,y}{d\,x}\right) = K\frac{d^3\,\alpha}{d\,x^3} - C\frac{d\,\alpha}{d\,x} \ldots\ldots\ldots\ldots (64)$$

if $C = E_s\,J$. Since the torque increases as α decreases, a negative sign is placed before the last term. Differentiating Equation (64) and taking account of Equation (6):

$$K\frac{d^4\,\alpha}{d\,x^4} - C\frac{d^2\,\alpha}{d\,x^2} - \frac{P\,x^2}{E}\left(\frac{1}{I_1} - \frac{1}{I}\right)\alpha = 0 \ldots\ldots\ldots\ldots (65)$$

which is the differential equation of elastic equilibrium of the cantilever subjected to a load, P, at the free end and slightly bent and twisted.

If the beam is incapable of developing a purely torsional resistance, C is equal to zero and, since, $I_1 = 2\,I_z$; $I = 2\,I_h$, and $I_f = I_z$ (approximately):

$$\frac{d^4\,\alpha}{d\,x^4} - R^2\,x^2\,\alpha = 0 \ldots\ldots\ldots\ldots (66)$$

in which,

$$R^2 = \frac{P^2}{E^2\,b^2\,I_z}\left[\frac{1}{I_z} - \frac{1}{I_h}\right]$$

which is Equation (27)* for the "F-beam".

If, on the other hand, the beam is of such a form that the contribution of the lateral bending of the flange to the twisting resistance is negligible, K is equal to zero, and the differential equation of elastic equilibrium for the J-beam is:†

$$\frac{d^2\,\alpha}{d\,x^2} + N^2\,x^2\,\alpha = 0 \ldots\ldots\ldots\ldots (67)$$

if,

$$N^2 = \frac{P^2}{E\,E_s\,J}\left(\frac{1}{I_1} - \frac{1}{I}\right).$$

A primitive of this differential equation may be found in terms of Bessel's functions,‡ or it may be integrated by means of a simple power series in the same manner as the author has integrated Equations (10)§ and (27). As an example of method, it may be interesting to sketch the process (for this rather simple case) in more detail than is given in the paper.

Assume that α can be expressed in a converging infinite series in ascending powers of x, thus: $\alpha = A_0 + A_1\,x + A_2\,x^2 + A_3\,x^3 + \ldots$. Then,

$$\frac{d^2\,\alpha}{d\,x^2} = 2\,A_2 + 2 \times 3\,A_3\,x + 3 \times 4\,A_4\,x^2 \ldots$$

Equation (67) then gives,

$$2\,A_2 + 2 \times 3\,A_3\,x + 3 \times 4\,A_4\,x^2 + \ldots + N^2\,A_0\,x^2 + N^2\,A_1\,x^3 + N^2\,A_2\,x^4 + \ldots = 0 \ldots\ldots\ldots\ldots (68)$$

* *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 13.

† The preceding analysis is essentially the same as that presented by A. and L. Föppl, "Drang und Zwang," First Edition, Vol. II, p. 350 *et seq.*

‡ See A. E. H. Love, "Mathematical Theory of Elasticity," 1920 Edition, pp. 425–427.

§ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 11.

If Equation (68) is to hold for all values of x, the method of undetermined coefficients requires that: $A_2 = A_3 = 0$; $A_4 = \frac{-N^2 A_0}{3 \times 4}$; $A_5 = \frac{-N^2 A_1}{4 \times 5}$; $A_6 = A_7 = 0$; and, in general, $A_n = \frac{-N^2 A_{n-4}}{n(n-1)}$.

All coefficients are thus expressed in terms of the two arbitrary factors, A_0 and A_1, which are the constants of integration for the second-order differential equation, and then:

$$\alpha = A_0 \left(1 - \frac{N^2 x^4}{3 \times 4} + \frac{N^4 x^8}{3 \times 4 \times 7 \times 8} - \dots\right) + A_1 x \left(1 - \frac{N^2 x^4}{4 \times 5} + \frac{N^4 x^8}{4 \times 5 \times 8 \times 9} - \dots\right) \dots\dots\dots\dots\dots\dots (69)$$

If it is desired to obtain the expression for the horizontal displacement, y, this may readily be done by means of Equation (7), thus:

$$E_s J \frac{d\alpha}{dx} = -P\left(y - x\frac{dy}{dx}\right) = -P x^2 \frac{d}{dx}\left(\frac{y}{x}\right)$$

from which,

$$\frac{1}{x^2}\frac{d\alpha}{dx} = -\frac{P}{C}\frac{d}{dx}\left(\frac{y}{x}\right), \text{ and } -y = \frac{C}{P} x \int \frac{1}{x^2}\frac{d\alpha}{dx} dx + B x \dots (70)$$

in which, B is a new integration constant.

If the value of α from Equation (69) is substituted in Equation (70);

$$-y = B x + A_0' \left(-x^3 + \frac{N^2 x^7}{2 \times 6 \times 7} - \frac{N^4 x^{11}}{2 \times 7 \times 8 \times 10 \times 11} + \dots\right) + A_1' \left(-1 - \frac{N^2 x^4}{3 \times 4} + \frac{N^4 x^8}{4 \times 5 \times 7 \times 8} - \dots\right) \dots\dots\dots\dots (71)$$

in which, $A_0' = \frac{1}{6}\frac{C N^2}{P} A_1$ and $A_1' = \frac{C}{P} A_1$. Equation (71) is substantially identical with Equation (11).*

From the relations arising in the evaluation of the constants, the author arrives at a value for the lowest critical load of,

$$P_k = 4.014 \sqrt{\frac{E I_3 E_s J}{L^4}}$$

Professor Prandtl has determined a value of,

$$P_k = 4.0126 \sqrt{\frac{E I_1 E_s J}{L^4}}$$

Noting that $I_3 = \frac{I_1}{1 - \frac{I_1}{I}}$, and that $\frac{I_1}{I}$ is always negligibly small in any case in which the "tipping" action of the beam is of any practical importance, the correspondence between the two results is practically exact.

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 11.

The author states* that he has extended his analysis to cover the case of flanged beams having an appreciable true torsional resistance, although this analysis is not included in the paper. It may be noted that the differential Equation (65) which applies to this case, may be integrated in precisely the same manner as the author has in Equations (10) and (27). The solution has been completely described by Professor Timoshenko,† who has also provided tables for use in the numerical application of his results. It should be observed that these tables are only intended for the case in which the load is assumed to be applied at the center of gravity of the cross-section ($n = 0$; $a = 0$ in the author's notation). Fig. 5 (*b*) is the only solution the writer has seen which takes into account the effect of the application of the load at other points than the centroid.

It is interesting to compare the critical load derived by Professor Timoshenko for a given case with that obtained from the author's curve. In the example under consideration,‡ the cantilever is a plate-girder section, with approximately the following properties: Span, 16 ft., 5 in.; depth, 1 ft., 7½ in.; $I = 670$; $I_1 = 6.8$; and the section is made up of 2⅜ by 2⅜ by ⅝-in. angles and 19½ by ⅜-in. web.

Then, from Fig. 5 (*b*),

$$a = 0, R L^3 = 15 = \text{(approximately)} \frac{P L^3}{E b \frac{I_1}{2}}$$

and,

$$P = \frac{15 \times 29\,000\,000 \times 19.5 \times 3.4}{197 \times 197 \times 197} = 3\,790 \text{ lb.}$$

The corresponding value obtained by applying Professor Timoshenko's tables is 3 840 lb. This indicates that the effect of the torsional resistance in a girder of such dimensions as the one considered is very slight compared to the resistance furnished by the lateral bending of the flanges, and since this is fairly evident from general considerations, the close correspondence of the two values constitutes a good check on the analysis.

S. TIMOSHENKO,§ ESQ. (by letter).‖—The author raises an important point in showing the necessity for investigations into the elastic stability of structures. This becomes especially important in light structures containing slender members, such as are used in railroad electrification. In such cases, a failure of the structure may occur, due to instability rather than to high stresses. Increasing the strength of the steel used does not increase safety in such cases, because the critical load for a slender member depends on the modulus of elasticity of the material, and not on its strength.

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 15.

† "Einige Stabilitäts Probleme der Elastizitätstheorie," *Zeitschrift f. Math. u. Phys.*, 1910, Vol. 58, pp. 337–385; and "Sur la Stabilité des Systèmes Elastiques," *Annales des Ponts et Chaussées*, Vol. III, pp. 496-566; Vol. IV, pp. 73-132; and Vol. V, pp. 372-412.

‡ "Einige Stabilitäts Probleme der Elastizitätstheorie," *Zeitschrift f. Math. u. Phys.*, 1910, p. 372.

§ Prof. of Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

‖ Received by the Secretary, April 19, 1929.

As an important example of elastic instability, the author discusses the problem of sidewise buckling of beams loaded in the plane of maximum rigidity. Two extreme conditions are considered: (1) Beams in which torsional resistance plays an important part in providing lateral stability; and (2) beams in which torsional resistance is due to bending of the flanges only. The first condition is fulfilled to a great extent in the case of beams of narrow rectangular cross-section. This problem has been discussed by L. Prandtl* and A. G. M. Michell.† Both arrived at the same equation for a cantilever as that given by the author,‡ namely:

$$P = \frac{4.014\sqrt{B\,C}}{L^2} \qquad (72)$$

in which, B is the flexural rigidity of the beam in a lateral direction; and C is the torsional rigidity of the beam. In the case of an I-beam, the problem becomes more complicated because not only torsional rigidity of the beam, but also the resistance of the flanges to bending, must be considered. The writer§ has shown that instead of Equation (72), the following equation must be used:

$$P_c = \frac{K\sqrt{B\,C}}{L^2} \qquad (73)$$

in which, K is a numerical factor depending on the proportions of the beam. Some values of this factor for various values of the ratio, $\frac{L^2}{a^2} = \frac{4\,C\,L^2}{B\,d^2}$, are as given in Table 2.

TABLE 2.—Numerical Values of the Factor, K, in Equation (73).

$\frac{L^2}{a^2}$	K.	$\frac{L^2}{a^2}$	K.
0.1	44.3	10	7.58
1	15.7	12	7.20
2	12.2	14	6.96
3	10.7	16	6.73
4	9.76	24	6.19
6	8.69	32	5.87
8	8.03	40	5.64

The factor, d, refers to the over-all depth of the beam. For greater values of $\frac{L^2}{a^2}$, the factor, K, can be calculated from the following approximate equation:

$$K = \frac{4.01}{(L-a)^2} \qquad (74)$$

It may be seen that, as the ratio, $\frac{L^2}{a^2}$, increases, the factor, K, approaches 4.01, and Equation (73) coincides with Equation (72). This means that for long

* "Kipperscheinungen," Dissertation, München, 1899.

† *Philosophical Magazine*, Vol. 48, 1899.

‡ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 12.

§ See *Bulletin*, Polytechnical Inst. at St. Petersburg, 1905–06; see, also, *Zeitschrift für Mathematik und Physik*, 1910, and *Annales des Ponts et Chaussées*, 1913.

I-beams the bending of the flanges becomes of secondary importance. The author also gives* without derivation some numerical results for a 12-in., 31.5-lb., steel I-beam supported at the ends and loaded at the middle. These results are in satisfactory agreement with results given by the writer.† For example, when $L = 20$ ft. and $a = 0$ (that is, the point of application of the load coincides with the centroid of the cross-section), the writer obtains‡ $f = 27\,000$ lb. per sq. in. This agrees with the ordinate in Fig. 7.§ It should be noted that the author determines the torsional rigidity by tests of small steel beams, although this quantity can be calculated with sufficient accuracy by using one of two approximate formulas proposed by the writer.‖

In conclusion, it should be noted that the question of the stability of a given configuration is connected with the investigation of the energy of the system in this configuration. The configuration is stable provided the corresponding energy is a minimum. On this basis of considering the energy of the system an approximate method for calculating the critical values of loads has been developed by the writer.¶

The method proved very handy for approximate solutions of stability problems, and in the case of sidewise buckling of beams it is simpler than the method used by the author, which is based on the investigation of the corresponding differential equation of equilibrium.

Furthermore, by using the energy method, problems can be discussed in which a member is subjected simultaneously to the "primary loading", and to a "secondary loading", and it can be proved that, in general, a member subject to deformation by the primary loading will not be so well adapted to resist the effects of secondary loading as if the first system of loading did not exist. Applying the method to struts submitted to axial compression (the primary loading), and lateral loading (the secondary loading), it can be proved that when all the lateral forces are in the same direction the "load-resisting function of the member as regards the secondary system is approximately proportional to 1 minus the ratio of the primary load or system to a similar load or system which would result in a condition of neutral elastic equilibrium.**

The problem of determining the center of twist in bending of non-symmetrical profiles, which is raised by the author†† in discussing bending of channel members, is also of practical importance, and has been extensively discussed in recent European literature.‡‡ The case of a cantilever in which the cross-section is a sector of a circle, or is bounded by two concentric arcs and two radii, is discussed in detail by M. Seegar and K. Pearson.§§ The case in which

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 16.
† *Transactions*, Am. Soc. C. E., Vol. 87 (1924), p. 1247.
‡ *Loc. cit.*, Table 5, p. 1255.
§ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 17.
‖ *Transactions*, Am. Soc. C. E., Vol. 87 (1924), p. 1249.
¶ *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 855.
** *Loc. cit.*, January, 1929, Papers and Discussions, p. 8.
†† *Loc. cit.*, p. 15, Fig. 5.
‡‡ See, A. E. H. Love, "Theory of Elasticity", IV Edition, 1927, p. 340.
§§ *Proceedings*, Royal Soc. (Lond.), (Series A), Vol. 96 (1920), p. 211.

the section is an isosceles triangle is treated by the writer.* The problem of determining the center of twist in the general case of a non-symmetrical profile can also be solved with sufficient accuracy, provided the thickness of the material is so small that the distribution of the shearing stress over it can be taken as uniform. This problem has been discussed in recent engineering literature by several authors.†

* *Proceedings*, Mathematical Soc. (Lond.), (Series 2), Vol. 20, 1922, p. 398.

† See R. Maillart, *Schweizerische Bauzeitung*, Vol. 77, p. 197; Vol. 79, p. 254; Vol. 83, pp. 111 and 176; C. Weber, *Zeitschrift für angewandte Mathematik und Mechanik*, Vol. 4, 1924, p. 334; A. Eggenschwyler, *Proceedings*, Second International Cong. of Mechanics, Zurich, 1926.

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PAPERS AND DISCUSSIONS

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STUDIES OF SHEAR IN REINFORCED CONCRETE BEAMS

Discussion*

By Hardy Cross, M. Am. Soc. C. E.

Hardy Cross,† M. Am. Soc. C. E. (by letter).‡—The author has treated so many aspects of his subject and has described them so fully that the writer has little hope of adding much by this discussion. Those who have been familiar with Professor Mylrea's extended researches in this field will value highly his recommendations.

The author has brought out clearly the importance of the stress trajectories, or traces of the planes of principal stress, in determining the direction at which cracks first appear. Within limits, however, the direction is necessarily a matter of chance. Until these cracks appear there are lines of tension sloping upward toward the end of a simply supported beam and lines of compression sloping downward toward the support. These may be regarded, respectively, as strings with a strut along the top supporting the load in tension and as tied arches supporting the load in compression all within the beam which supports the load in flexure. The arch action and catenary action which designers are constantly discovering in concrete beams, are, therefore, perfectly real; but they are no more real than they are in timber beams or steel beams; they are different aspects of one and the same phenomenon, and not separate things at all.

As cracks occur along the lines of tension the arches carry more and more of the load. It would now seem as if these arches could not fail in shear; but elongation of the main reinforcement and diagonal shortening of the concrete and especially bond slip—all somewhat similar to rib-shortening as usually conceived in arch design—throw secondary bending into these remaining arches and the thrust lines are now inclined to the arches.

* Discussion of the paper by T. D. Mylrea, M. Am. Soc. C. E., continued from April, 1929, *Proceedings*.

† Prof., Structural Eng., Univ. of Illinois, Urbana, Ill.

‡ Received by the Secretary, April 19, 1929.

The writer thinks that such conceptions as these are rarely satisfactory. They do, however, have some value in bringing attention to the importance, in dealing with a brittle material, of phenomena which are often thought of as secondary. They lead here to a word of caution in dealing with beams with sloping faces, because of these secondary effects. Because concrete is so brittle, it should be fully reinforced.

There are four methods of studying structural design: Analysis, tests, experience with structures in service, and imagination as to possible methods of failure. In reinforced concrete no one of these alone can be called either the proper method or the scientific method of study; all have been at times neglected or over-emphasized; all have proved at times powerfully illuminating and, at other times, deceptive and discouraging.

Perhaps the chief value of the paper is in correlating these rational, semi-rational, and empirical viewpoints and especially in suggesting the importance of secondary modes of failure, such as crushing at bends or splitting from hooks, the importance of which so many designers fail to realize.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE GRAPHICAL SOLUTION OF A CORRELATION TABLE

Discussion*

By B. F. Jakobsen, M. Am. Soc. C. E.

B. F. Jakobsen,† M. Am. Soc. C. E. (by letter).‡—The author is to be complimented for submitting his graphical method for determining the coefficient of correlation, r, and its probable error. The method presented is certain to appear at a disadvantage, due to the small scale made necessary in reproducing it in a paper. The small scale gives the impression of a complexity, that probably would not occur if the scale was large, as it would be, in actual use. Fig. 12§ gives a clear illustration of the entire problem, including the analytical or arithmetical solution for those who may prefer that. Mr. Arne Fisher has pointed out‖ that, when the probable error cannot be determined, no great importance can attach to the result, since the probable error may be as large, or even larger, than the result itself.

The writer believes engineering will benefit, if engineers will acquire the habit of analyzing their "facts" by appropriate mathematical methods, such as those outlined in this paper. The truth is that the writer leans somewhat to the opinion that "facts" may not become "facts" until they are so analyzed by proper methods, or, at least, that they are not useful facts in a scientific sense, until analyzed and properly qualified.¶

The main problem is to determine the probable error, for it gives information as to the reliability of the observations and the theory. If the probable error is larger than the actual errors likely to have been made in measuring, it signifies that the equation assumed does not represent the phenomenon

* Discussion of the paper by Weston S. Evans, Esq., continued from May, 1929, *Proceedings*.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, April 8, 1929.

§ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 55.

‖ *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 83.

¶ See Karl Pearson, "Grammar of Science" (1911), and the various popular writings of the great French mathematician, Henri Poincaré, "Science and Hypothesis," "The Value of Science," "Science and Method," and "Dernieres Pénsées".

under investigation with sufficient accuracy. For simple problems the method of least squares suffices, but for more complicated phenomena, correlation must be used.

When Kepler was studying the observations made by Tycho Brahe, in order to determine the orbits of the planets, he assumed at first a circular path. He found "errors" of 7′ and 8′, but his confidence in Tycho's measurements was so implicit that he refused to believe that the actual errors of observations were that great. He then tried an elliptic orbit, assuming the sun to be located in one focus, and found "errors" of only 1′; this led to the discovery of the now famous Kepler's laws of planetary motion, from which Newton derived his law of gravitation.

The writer has analyzed the results of tests on the Stevenson Creek Dam and has found for the arch at Elevation 30, that p, the water load carried by the arch, was 9.80, and its probable error, ± 0.50; for a factor, k, determined by the shrinkage, he found k equal to -681 ± 480.* The probable errors show that the value found for p is quite reliable, while the value for k is much less so.

Professor Karl Pearson has shown† that experience does not and cannot prove causation, which is an assumption supplied by Man and then projected into the world of phenomena; but experience does show correlation. The coefficient of correlation with its probable error makes it possible to determine the degree of correlation and to compare one experience with another. A great many important problems, now generally drowned in a mass of words, can be attacked intelligently and effectively only by means of the coefficient of correlation. Such problems as the effect of strikes on wages, the effect of concentration of industry in large establishments on wages and working conditions, etc., can be unraveled only by appropriate mathematical treatment.‡

Every engineer must often have been vexed or amused when reading the voluminous opinions offered by successful business men, politicians, or chiefs of Police Departments, on the causes of automobile accidents. This problem might be profitably studied by a few men properly trained and equipped, and such a study would hinge largely on the determination of the various coefficients of correlation.

Leonardo da Vinci wrote:§ "He who scorns the certainty of mathematics will not be able to silence sophistical theories which only end in a war of words." Although written more than four centuries ago, and in spite of considerable progress made in the meantime, this statement applies as if it were written only yesterday.

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* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1243.

† "Grammar of Science," by Karl Pearson, 1911.

‡ See, for example, "Law of Wages," by Prof. Henry Ludwell Moore, 1911.

§ "A Short History of Science," by W. T. Sedgwick and H. W. Tyler, Macmillan Co., 1927.

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PAPERS AND DISCUSSIONS

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EXPERIMENTS ON DISCHARGE OVER SPILLWAYS AND MODELS, KEOKUK DAM

Discussion*

BY MESSRS. B. F. JAKOBSEN, J. B. SPIEGEL, DANA M. WOOD, S. P. WING, E. A. RUDULPH, AND C. H. PIERCE.

B. F. JAKOBSEN,† M. AM. SOC. C. E. (by letter).‡—These tests are of particular interest, because they furnish information regarding the discharge coefficient for heads up to 11 ft. for the Keokuk spillways. A head of 1 ft. on the model corresponds to 11 ft. on the Keokuk spillways, and the agreement between the model and the actual structure is quite close.§ It would be interesting to have some probable errors calculated for a number of these tests and to determine the coefficient of correlation between the tests on the model and those on the actual structure.

The tests suggests that the discharge coefficient of the Francis formula, $Q = C\,L\,H^{1.5}$, is not a function of the head, but of some ratio of, or relation between, the length and the head. It is possible that the authors may be able to derive some such relation from the tests already made. If not, additional tests could be made, for example, by introducing a portable pier at Keokuk and also in connection with the model, so as to be able to vary the ratio between length and head. If the authors could establish some such relation, it would be of considerable practical value, since the discharge coefficient, C, varies from 3 to 4.

With reference to Figs. 12‖ and 13,¶ it would be instructive to have the value of C ploted for a number of vertical sections, beginning as close as possible to the piers.

* This discussion (of the paper by Floyd A. Nagler and Albion Davis, Members, Am. Soc. C. E., published in February, 1929, *Proceedings*, but not presented at any meeting of the Society), is published in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, April 10, 1929.

§ *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 338, Fig. 20.

‖ *Loc. cit.*, Papers and Discussions, p. 326.

¶ *Loc. cit.*, p. 327.

It appears that the authors use the term, "actual velocity", when they mean "maximum velocity". For example, they state:* "The contours in Figs. 12 and 13 * * * show the actual velocity of the water which was measured throughout the nappe."

From what follows in the text, the writer infers that the reference is to the maximum velocity, which, however, was not measured, but calculated from measurements. As the writer understands it, the horizontal velocity is as actual as the velocity in any other direction. The influence of the shape of the piers, as noted by the authors,† should also be carefully studied.

The tests appear to have been well conceived and carefully made, despite considerable difficulties, and the authors deserve the thanks of the profession for presenting their methods and results.

J. B. SPIEGEL,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—The authors have made a valuable contribution to engineering knowledge by their presentation of the results of the experiments and tests of weir discharge performed at the Keokuk Dam. After reading the paper one can readily understand the reasons that delayed its appearance until five years after the experiments were finished. Those who are constantly studying the behavior of surface waters and who realize how applicable this information is to numerous problems that require solution will be pleased to learn that they may now select coefficients for high-head weir discharges with greater confidence. Uncertainty has always lurked in the minds of engineers concerning the influence of the opening of neighboring gates on individual weir discharges.

Fig. 14‖ reveals a relationship that is indeed interesting, but leads one to wonder just where the central-weir velocity curves would plot for the condition of nine, eleven, thirteen, etc., gate openings. Apparently, the greater the number of openings the closer the results approach the curve of velocities corresponding to the head below pond level. It is appreciated that a greater number of openings at the time these observations were taken probably would have seriously affected the efficient operation of the power plant and would have been wasteful of water.

Since practically all velocity measurements made by the U. S. Geological Survey are obtained with Price current meters, a statement of results as compared with the registered or adjusted values of the Ott meter would be of interest to its engineers. Velocities of more than 12 ft. per sec. are rarely exceeded in ordinary stream-gauging work and no appreciable wear on the meters is noted except when they are used in waters heavily charged with silt.

The Keokuk Dam is located at a section that carries the discharge from about 119 000 sq. miles of drainage area. The component areas have not been studied comprehensively or continuously. Therefore, no reliable estimate of flow can be obtained, except by use of the Keokuk records and these experiments assure the accuracy of these records for at least the 16-year period of

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 325.
† *Loc. cit.*, p. 326.
‡ Dist. Engr., U. S. Geological Survey, Topeka, Kans.
§ Received by the Secretary, April 15, 1929.
‖ *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 328.

operation of the plant. Gauge height observations and some discharge measurements on the Mississippi River at this location prior to that period and extending back to the late Seventies afford a record of fifty years or more, which is a valuable contribution to the study of the perplexing Mississippi River flood control problem.

The recent *Bulletin** issued by the State Engineer of California and the conclusions of the Vermont Flood Commission† indicate that engineering thought is directed toward the study of the feasibility of co-ordinating water-power development and flood control. The perfection of discharge formulas for application in such projects, on which weirs often play an important part, is timely and welcome.

DANA M. WOOD,‡ M. AM. SOC. C. E. (by letter).§—This paper brings forcibly to consideration the behavior of current meters when subjected to angular flow conditions.

The ideal meter would be one which, when held rigidly in a horizontal plane and at right angles to the measuring section, would correctly measure that component of the velocity normal to the section. It is to be noted that in the Keokuk experiments, the meter was allowed to swing in a horizontal plane to meet the direction of the average current striking it, the angularity being determined by an indicating device. The meter was not permitted to swing in a vertical plane, except in two special tests. Therefore, the effect of variations in vertical angularity and of minor departures from the average or predominating horizontal angularity, at any point of measurement, was taken account of only in the standard behavior of the particular meter used under the composite conditions existing, with a standard correction obtained from Fig. 9.‖

For some time it has been known that certain of the standard makes of current meters over-register, and that others under-register, the true normal velocity when held rigidly in a normal horizontal position and when currents are oblique to the meter in either a horizontal or vertical plane, or a combination of each. To what extent in the Keokuk experiments correction for horizontal angularity only accounts for all possible over or under-registry, is not made clear in the paper. It hardly seems possible that Fig. 9 is applicable to all stations, especially those near the piers.

Furthermore, there is usually not only a difference in the error of registration resulting from horizontal and vertical angularity of currents, but also a difference from currents hitting the meter from the left and from those hitting it from the right. Variations of methods of suspension, and size and shapes of weights, if they are used, affect meter ratings and registration, as has been well proved by the Water Resources Branch of the U. S. Geological Survey. It is open to question whether one standard laboratory correction curve adequately covers variations actually occurring during the measurements.

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* *Bulletin No. 14*, "The Control of Floods by Reservoirs."

† *Engineering News-Record*, p. 72, Vol. 102, No. 2.

‡ Hydr. Engr., Stone & Webster Eng. Corporation, Boston, Mass.

§ Received by the Secretary, April 25, 1929.

‖ *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 320.

The matter of whether or not full consideration has been given to all these questions, or whether their effect has been determined to be negligible, in view of other errors of measurement possible, is not made clear in the paper.

It is very important to select the proper type and make of current meter when used for experiments as described. While the Ott type of meter may have been best adapted to the Keokuk conditions, such general statements in favor of its selection as* "it measures the normal component of angular currents more accurately than other types", might be amplified for clearness. If the meter is allowed to swing in either a horizontal or vertical plane, and its axis assumes a position parallel to the average stream line, the statement would refer to a plane "normal to the meter" rather than "normal to the measuring section", although the latter is the desired velocity. Accurate results can be obtained only when the direction which the axis of the meter assumes can be reliably ascertained. There may well be a set of conditions in which another type of meter might be better suited for determining the velocity normal to the measuring section even if the Ott type proved best at Keokuk.

The authors state† that "with the 3-blade propeller it [the Mensing-Ott meter] had characteristics in measuring angular currents which were superior to the others". In view of the paucity of experimental data on the behavior of current meters under conditions of angular velocity, would it not be well if the authors developed the basis for this statement more fully?

With the lower actual velocity and depth conditions in the model tests, is it safe to assume that, because of the apparent verification of Pitot tube checks, the limit of accuracy of the field tests was within 2 per cent? In view of the care taken in all the work and the study given to the many complicated problems, the conclusion may be correct, but the writer would like to see some discussion from others who have conducted laboratory and field tests on these questions.

S. P. Wing,‡ Assoc. M. Am. Soc. C. E. (by letter).§—Practical hydraulics, being largely an empirical science and one in which the required coefficients have been determined for the most part from tests on structures of a size far smaller than those used in practice, has been handicapped for the lack of adequate design data. At the time this paper was published, the writer was endeavoring to fix the coefficient of discharge for a large overflow dam. By making use of the data presented he was enabled to make a direct saving in the size of his proposed flood-gates. The work of the authors is of particular value, in that experiments were made on both a model and on the full-sized structure.

Although the theory of models is well established, certain factors cannot be adequately reduced to scale and tests of the full-scale structure remain the final criterion. The high degree of correlation obtained by the author, between the results of the model and those of the full-scale structure, means that future tests on similar structures can be confined to those made on the cheaper models.

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 308.

† *Loc. cit.*, p. 315.

‡ Designing Engr., Constr. Dept., British Columbia Elec. Ry., Ltd., Vancouver, B. C., Canada.

§ Received by the Secretary, April 30, 1929.

The tests on the model at high heads with the coefficients, as shown in Fig. 19,* are of special interest. These reach the value of 4.15 and the curve gives no indication of flattening. The upper limit should be reached when the depth of overflow arrives at a point at which the nappe jumps clear of the face; it would be interesting to know what this limit is. It would also be of interest to know at what point a vacuum begins to form below the nappe.

The economic design of the crest of an overflow dam has two main controlling factors: First, the design should be such that the coefficient of discharge will be as large as possible; and, second, such that no vacuum will take place under the overfalling sheet. With present information as to the shape of the lower nappe, based on Bazin's experiments and as applied in Creager and Justin's Hydro-Electric Handbook, if vacuum is to be prevented, the relation of the depth of overflow with respect to the curve of the dam crest must be such that the coefficient of discharge is limited to about 3.9 or 4.0 for an unobstructed crest. It would be of interest to know whether this limit exists in fact, as the profile and the cost of an overflow dam are controlled largely by this consideration.

In applying the results of these tests to other structures, it becomes necessary to find the scale at which the proposed strucure most closely resembles that on which the tests were made, and then to make use of this scale in determining the proper discharge coefficient. This fact occasionally has been overlooked, data being published as to the value of the coefficient for certain profiles or for certain depths of flow without giving the full dimensions of the structures and the depths of flow at which the tests were made. In fitting a proposed profile to a test profile to find the scale relationship, probably most weight should be given to the fit of the up-stream face. Small differences at that point have a large effect on the coefficient of discharge as was shown in the model experiments† made at Cornell University. Additional experiments are needed as to the effect of varying the down-stream profile.

The authors have shown that results with a high degree of accuracy can be obtained from relatively inexpensive hydraulic models. The writer hopes that they and others will make additional tests to obtain coefficients for the lack of which, hydro-electric designers now must use too much guesswork. A foot of head frequently is worth $100 000, and the economic design of a flow line deserves better data than are now available.

E. A. RUDULPH,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—This paper is particularly timely in view of the growing interest now being manifested by hydraulic engineers in the use of models for the solution of practical hydraulic problems.

The great difficulty and high cost of making accurate measurements of the large quantities of water and high velocities usually encountered in hydraulic structures, have resulted in a scarcity of conclusive information to demonstrate the agreement between model and full-scale measurements.

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 335.
† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1199.
‡ Engr., Stone & Webster Eng. Corporation, Boston, Mass.
§ Received by the Secretary, May 1, 1929.

In consequence, engineers have hesitated to adopt the results of experiments on models as a reliable measure of full-scale hydraulic conditions.

The close agreement between model and full-scale measurements obtained through the accurate and painstaking work done at Keokuk should go far toward establishing confidence in results obtained from model tests. Moreover, in view of the large-scale model used by the authors, and the greater possibility of error in the experiments on the full-sized structure, it is possible that the most reliable discharge coefficients of the Keokuk Spillway are those determined by the model test.

An essential requirement in any hydraulic model test is that true similarity between model and full-scale conditions be attained. The same reasons which cause engineers to hesitate in accepting model-test results frequently cause them to select, when such experiments are attempted, the largest scale for the model that laboratory facilities will permit. In so doing the true hydraulic similarity between the model and its full-scale prototype is often overlooked. The following example from the work on the Keokuk model illustrates this point.

While the head on the crest of the Keokuk Spillway was measured at points where little or no velocity normal to the dam existed and, therefore, required no correction for the velocity of approach, the authors do not indicate that the results obtained from the model were corrected for velocity of approach in the test flume. With all three gates open this velocity would have corresponded to about 1.2% of the normal head of 1 ft. on the crest of the model. Applying this correction to the coefficient obtained for three gate openings, it is reduced to approximately 3.75, a value somewhat more reasonable in the light of Tests 4 and 5 on the full-sized structure. With only one gate on the model open, the velocity of approach becomes so small as to be negligible, and no correction is required. If a model of smaller scale had been used, a greater number of gates could have been accommodated in the 10-ft. width of the test flume and the approach conditions to the spillway could have been made to simulate more truly those of the full-sized structure. The head measurements could then have been made in a location similar to those for the actual spillway, and no velocity of approach correction would have been involved.

Greater precision of measurements will naturally follow from the use of large-scale models, assuming adequate laboratory facilities for measuring accurately the larger quantities of water used. Moreover, the comprehensive study of flow conditions within the overfalling sheet made by the authors would require a large-size model. Ordinarily, however, entirely satisfactory quantitative determinations can be obtained from models of much smaller scale than were used for the Keokuk tests.

Experiments in European laboratories have demonstrated that as long as the velocity remains sufficiently high to insure a turbulent or rolling condition of flow, the laws of hydraulic similarity continue to apply. Any difference in results obtained by models of different scales, therefore, will be due to the relative frictional resistance to flow. The discharge of a spillway is affected by frictional resistance of the surface of the structure only in

the region of low velocity up stream from, and a short distance over, the crest. Below this point the surface of the overfalling sheet falls below the critical depth, and frictional resistance no longer has any effect on the discharge. For this reason the effect of the relative roughness of spillway models is negligible, a fact demonstrated by the authors by comparing the discharge coefficients obtained from sanded and smooth-surfaced models. The use of a small model, therefore, will affect the accuracy of the results only in so far as it affects the accuracy of the head and discharge measurements. Aside from securing true hydraulic similarity the use of small-scale models has the added advantage of lower cost and increased facility of testing.

The reliability of the model tests having been established by comparison with the test on a full-sized structure, the importance of making model experiments in the design of hydraulic structures is strikingly demonstrated by the experiments performed by the authors, which show the possibility of materially increasing the discharge capacity of the spillway by the adoption of properly shaped piers.

C. H. PIERCE,* M. AM. SOC. C. E. (by letter).†—Ratings of hydraulic structures by means of laboratory measurements on small scale models form one of the most interesting and fascinating subjects in the field of hydraulic research. Experiments and laboratory tests of models have become fairly common in occurrence, but rarely is the opportunity available for comparison of laboratory experiments and tests with the results of measurements made on the full-sized structure under operating conditions. Therefore, the experiments described in this paper should be of especial value.

Some of the earlier tests on models which have come to the writer's attention indicated that the requirements of mechanical and hydraulic similitude were not always fully complied with, and, consequently, the results of the tests were not conclusive. It is possible also that all the laws of hydraulic similitude may not as yet be established or understood. For example, it is only recently that the foremost experimenters in this subject have fully comprehended the time relation for motion picture exposures required for an undistorted time scale from the model.

In 1918, Benjamin F. Groat, M. Am. Soc. C. E., gave a statement‡ of the "Theory of Hydraulic Models", and showed the derivation of some of the fundamental relations which exist between the full-sized object and the model. More recent expositions of these relations have been made by Professor Hubert Engels and others.§

A condensed restatement of the known laws of hydraulic similitude may be permissible. For a model built to a scale relation of $1 : k$, the coefficients to be applied to the model features in order to transfer them to the full-sized structure are as follows:‖

* Care, Charles T. Main, Inc., Boston, Mass.

† Received by the Secretary, May 31, 1929.

‡ "Ice Diversion, Hydraulic Models, and Hydraulic Similarity," *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), pp. 1108–1190.

§ See "Hydraulic Laboratory Practice," Edited by John R. Freeman, Past-President, Am. Soc. C. E.; and pub. by the Am. Soc. of Mech. Engrs., January, 1929.

‖ Arranged from notes of lectures given by Prof. Dr.-Ing. Theodor Rehbock, at Mass. Inst. Tech., March, 1929.

Coefficient.	Kind of Quantities to Be Transferred.
$k^{-0.5}$	Number of revolutions of water-wheels (pumps, turbines); number of motion picture exposures for undistorted time scale.
k^{0}	Constants; ratios (obstruction ratios, flow ratios); relative slopes; unit weights; accelerations.
$k^{0.5}$	Time; velocities.
$k^{1.0}$	Lengths; widths; heights; absolute slopes; velocity heads; friction heads; stresses; strains; wind pressure; towing resistance per unit of area; temperature changes through frictional resistance.
$k^{1.5}$	Discharge volume per unit of time and of width.
k^{2}	Areas (cross-sections).
$k^{2.5}$	Discharge quantities per unit of time for the entire stream.
k^{3}	Cubic contents; water volumes; mass forces (weights, impact, towing forces, wind pressures, reactions).
$k^{3.5}$	Work done per unit of time; magnitudes of movements (impulses).
k^{4}	Movements; work; energies (potential and kinetic energies).

The velocity with which the water approaches the crest is one of the major factors which determine the discharge equation of the spillway, and it is essential that the approach channel of the model shall be constructed so that the relation of velocity heads will conform with the law of similitude. Another important consideration is that the relative slopes of the water surfaces shall be in conformity. The fulfillment of this latter requirement of itself will assure the proper degree of smoothness of the model.

It would be manifestly impossible to reproduce, in an hydraulic laboratory of an engineering school, all the diverse conditions pertaining to the 119 spillway sections which extend across the Mississippi River from the Illinois shore to the power house at Keokuk, Iowa. Certain typical conditions can be evaluated, however, and general conclusions drawn from them. This appears to have been the method of attacking the problem at Keokuk, with respect to the current-meter measurements and the model tests.

Eight current-meter measurements were made of discharge over one spillway.* These measurements may be arranged in five groups as follows:

(*a*) No adjacent gates open (Measurements Nos. 1, 2, and 8).
(*b*) One adjacent gate open (Measurement No. 3).
(*c*) Two adjacent gates open (Measurements Nos. 4 and 5).
(*d*) Four adjacent gates open (Measurement No. 6).
(*e*) Six adjacent gates open (Measurement No. 7).

Because of the small range in head between the highest and lowest measurements, (0.37 ft. between the head of 10.99 ft. for Measurement No. 7, and 10.62 ft. for Measurement No. 5), it is necessary to plot them at an extended scale in order to analyze the results graphically. This has been done in Fig. 27, in which logarithms of head have been plotted against logarithms of discharge, using ordinary cross-section paper. The results of this plotting are fairly consistent. Measurement No. 7, which probably had the greatest velocity head on account of the large number of open gates, plots highest on the discharge curve. Group (*a*), with no adjacent gates open, is consistent

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 318, Table 1.

with respect to location of the group, although the individual measurements differ somewhat among themselves. Measurement No. 3, with one adjacent gate open, shows a slightly larger discharge coefficient than the average of Group (*a*), which is as might be expected. The straight line defined by Group (*c*) shows a still larger coefficient, due to the greater velocity head. At first glance, Measurement No. 6, with four gates open, does not appear to be consistent, until it is noted that the four gates were all on one side of the section, and, consequently, the velocity head at the measuring section was probably about the same as for the condition of one adjacent gate open, as in the case of Measurement No. 3. Measurement No. 7, with six adjacent gates open, and with three gates on each side of the measuring section, shows the highest coefficient of all, about 2.839, with 1.628 as the exponent of the head.

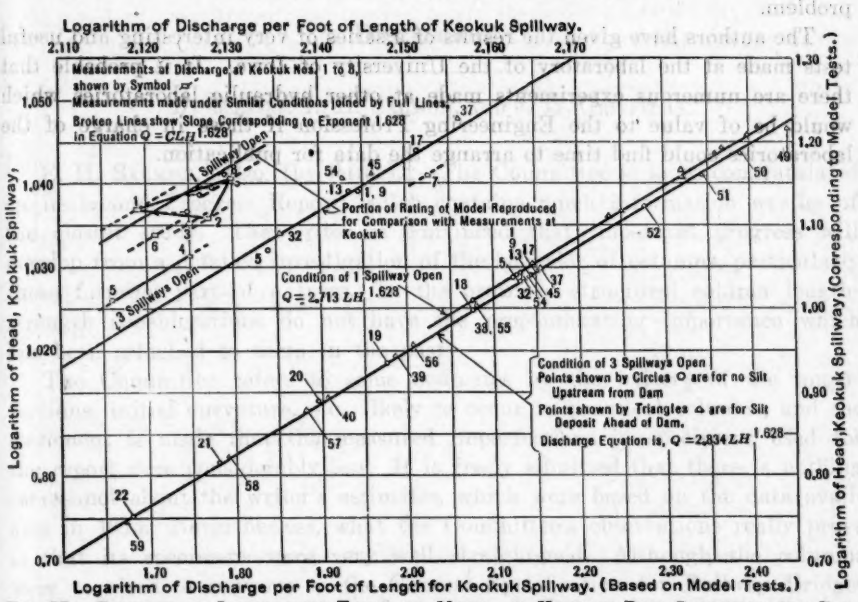

Logarithm of Discharge per Foot of Length of Keokuk Spillway.
Measurements of Discharge at Keokuk Nos. 1 to 8, shown by Symbol
Measurements made under Similar Conditions joined by Full Lines.
Broken Lines show Slope Corresponding to Exponent 1.628 in Equation Q = CLH 1.628
1 Spillway Open
3 Spillways Open
Portion of Rating of Model Reproduced for Comparison with Measurements at Keokuk
Condition of 1 Spillway Open
Q = 2.713 LH 1.628
Condition of 3 Spillways Open
Points shown by Circles ○ are for no Silt Upstream from Dam
Points shown by Triangles △ are for Silt Deposit Ahead of Dam.
Discharge Equation is, Q = 2.834 LH 1.628
Logarithm of Head, Keokuk Spillway,
Logarithm of Head, Keokuk Spillway, (Corresponding to Model Tests.)
Logarithm of Discharge per Foot of Length for Keokuk Spillway. (Based on Model Tests.)

FIG. 27.—RESULTS OF LABORATORY TESTS ON MODEL OF KEOKUK DAM COMPUTED FOR DISCHARGE PER FOOT OF LENGTH OF KEOKUK SPILLWAY. RATIO, $n = 11$.

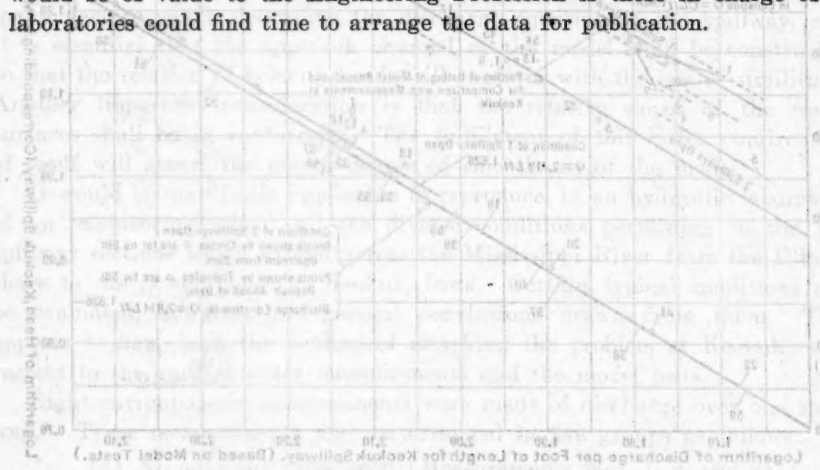
In studying the rating of the model, it will be seen that for the condition of three spillways open, all the measurements made under that condition plot very close to a line the equation of which is $Q = 2.834\ L\ H^{1.628}$. The condition of silt ahead of the dam appears to have given results not appreciably different from the condition of no silt deposit. For the condition of one spillway open, the measurements define a line having the equation, $Q = 2.713\ L\ H^{1.628}$.

It should be noted that the small group of measurements with one spillway open, which are reproduced at the enlarged scale between the heads having logarithms of 1.035 and 1.041 plot somewhat larger than the average curve for that condition. It is believed, however, that the average curve is more dependable than individual measurements or any one group of observations.

For those who desire to use $H^{\frac{3}{2}}$ and a coefficient varying with the head instead of the exponential equation as derived from a straight-line log-diagram, the writer would suggest that it is easy to recast the equations mentioned into the form, $Q = C_{var.} L H^{\frac{3}{2}}$. The writer believes that logarithmic plotting affords the simplest and most reliable means of analyzing and evaluating many types of experimental data, especially those obtained from hydraulic experiments.

It is gratifying to note the considerable number of engineering schools and engineering experiment stations which now have equipment for conducting hydraulic experiments on a large scale. These laboratories, with the support and encouragement of engineers who are interested in hydraulic research, can be developed so as to provide ample facilities for the study of any hydraulic problem.

The authors have given the results of a series of very interesting and useful tests made at the laboratory of the University of Iowa. It is probable that there are numerous experiments made at other hydraulic laboratories which would be of value to the Engineering Profession if those in charge of the laboratories could find time to arrange the data for publication.



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PROGRESS REPORT OF THE SPECIAL COMMITTEE ON STEEL COLUMN RESEARCH

Discussion*

BY MESSRS. E. H. SALMON, AND J. H. GRIFFITHS.

E. H. SALMON,† ESQ. (by letter).‡—The Committee is to be congratulated on its second Progress Report, which contains much information worthy of the closest study. The writer is convinced that important progress will develop from a detailed investigation of the behavior of columns, particularly those forming part of a truss. In the ordinary structural column length-strength considerations do not have the preponderating importance which has been attached to them in the past.

The Committee refers to some estimates by the writer§ of the imperfections, initial curvature, etc., likely to occur in practical columns, and the statement is made that the measured imperfections in specimens used for the report were considerably less. It is freely admitted that there is nothing sacrosanct about the writer's estimates, which were based on the data available in 1915. Nevertheless, what the Committee's observations really prove is that its specimens were very well straightened. Although the columns were required to conform to the General Specification for Railway Bridges of the American Railway Engineering Association, it would, perhaps, be rather natural if a little extra care were taken with members intended for such a special purpose. However this may be, it is of interest to compare the Committee's suggested formula, $e' = \frac{l}{1\,500}$, with earlier observations. The writer has reached the conclusion that there is no evidence to show that the initial deflection is a function of $\frac{l}{r}$ rather than of l, with which the Committee seems to agree. The larger diagram in Fig. 40 is the original from

* Discussion of the Progress Report of the Special Committee on Steel Column Research continued from May, 1929, *Proceedings*.

† Clark & Standfield, London, England.

‡ Received by the Secretary, April 5, 1929.

§ *Proceedings*, Am. Soc. C. E., February 1929, Papers and Discussions, p. 433.

which the rule, $e' = \frac{l}{750}$, was deduced; a line representing the Committee's suggestion, $e' = \frac{l}{1\,500}$, has been added. It is found by counting that of the 185 plotted points, 7 lie outside the line, $e' = \frac{l}{750}$, and 78 (42%) lie outside the line, $\frac{l}{1\,500}$. It is submitted that this number is much too large to be fortuitous, and that, in practice, it would not be feasible to rely on securing a crookedness as slight as $\frac{l}{1\,500}$. Examining Fig. 37* in the same way: Of the 39 plotted points, 5 of them, or 13%, are outside the line, $e' = \frac{l}{1\,500}$. It is to be observed that most of these specimens were about 100 in. long, and that in the fewer long specimens, the initial deflection was small, whereas in Fig. 40 the percentage in which $\frac{l}{1\,500}$ is exceeded, is greater for the greater lengths.

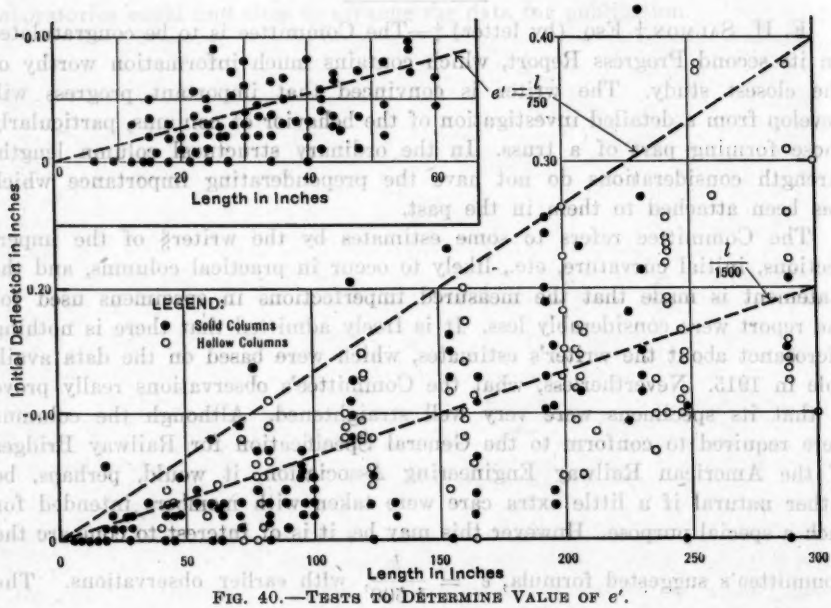

FIG. 40.—TESTS TO DETERMINE VALUE OF e'.

Taking Fig. 37 in conjunction with Fig. 40, it appears to the writer that, with the data available, the estimate, $e' = \frac{l}{750}$, is not unreasonable. Nevertheless, what he would like to see would be the measurement (as suggested in the Committee's previous report†) of a large number of actual mem-

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 438.
† *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1537.

bers (not test specimens) as fabricated ready for erection, made at different yards, and classified as to size and shape; that is, small single bars differentiated from large built-up members. The cost should not be excessive, and if from these data, the Committee could fix a standard which might pass inspection in the shops, he conceives that one debated point in column design would have been settled. This is a matter in which the large steel fabricating companies might co-operate usefully to a common advantage.

The Committee finds* that the variations in E and its effects are of much less importance than predicated by the writer. This remark appears to have reference primarily to the average value of E for the column as a whole; that is, as used in the secant formula. Perhaps a little misunderstanding has arisen here, for the writer fully agrees with this conclusion; in fact, he has shown† that a $\pm 10\%$ variation in E made no practical difference in the value of EI, provided the average value was used for E. It is of importance, however, to recognize the effect of a difference in the value of E for the two flanges of a built-up member, for example, in shifting the center of resistance away from the geometrical center of the cross-section. It is easy to prove that a $\pm$ 10% variation in E shifts the center of resistance, $\frac{h}{20}$, from the geometrical center, where h is the distance between the centroids of the two flanges.

With the data available prior to 1915, the writer estimated the probable variation in E as ± 10 per cent. The Committee finds for the built-up columns a range of 10% (— 6.6 to + 3.8%).‡ It is quite probable that the previous estimates for the values of E in compression are inaccurate. The determination of E in compression experiments is not easy. Strain-gauge measurements are really not very suitable, and the effect of flexure is probably always present. The writer based his estimate chiefly on Bauschinger's§ experiments on wrought-iron specimens, which were very carefully made (range, 22½%), and on fifteen experiments made at Watertown Arsenal‖ on mild steel specimens from 1908 to 1912 (range, 16½%). Fidler,¶ who first insisted on the importance of the variation in E, estimated the range as 23% for wrought-iron specimens. In Christie's** compression experiments, the range was much higher: Wrought iron = 56½%; mild steel=45.7%; and hard steel = 51.2 per cent. There is little doubt that these high values are due to flexure. Modern experiments indicate that E is the same for tension and compression (see, for instance, Dalby††), and it is noteworthy that the observed variation in tension experiments, which are much easier to make,

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 421.

† "Columns," by E. H. Salmon, Oxford Technical Publications, p. 34.

‡ *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 413.

§ "Zerknickungs-versuche," *Mitt.* a. d. Mech. Tech. Lab. d. k. Tech. Hochschule in München, Heft 15, Mitt. XVIII, p. 11.

‖ Reports of Tests of Metals, Washington, D. C., 1909–13.

¶ "The Practical Strength of Columns," *Minutes of Proceedings*, Inst. C. E., Vol. LXXXVI, p. 261, 1886; and "A Practical Treatise on Bridge Construction," Lond., First Edition, 1887.

** "Experiments on the Strength of Wrought-Iron Struts," *Transactions*, Am. Soc. C. E., Vol. XIII (April, 1884), p. 85; (August, 1884), p. 253.

†† "The Mechanical Properties of Steel," *Minutes of Proceedings*, Inst. C. E., Vol. CCXXI, p. 21, 1925–26.

seldom exceeds ±5% (Christie's values give 19.1% for tension experiments). Nevertheless, careful analysis of the Committee's figures for the columns tested (Tables 25* and 26†) indicate that this variation may be considerably exceeded in particular cases.

Taking the assumed average value, $E = 30\,000\,000$ lb. per sq. in., it will be seen that for the built-up columns (Table 25) in the 10 observations on 20-in. gauge lengths, the variation in E reaches or exceeds 5% three times, the maximum variation being from —8.7 to +5.0%, a range of 13.7%; and in 33 observations on 8-in. gauge lengths, the variation reached or exceeded 5% seven times, the maximum variation being from —7.7 to +7.3%, a range of 15 per cent. In Table 26 for the H-beams, the values are more uniform, but in 52 observations on 8-in. gauge lengths, a variation of 4% was reached or exceeded on nine occasions, the maximum variation being from —5.7 to +7.7%, a range of 13.4 per cent. In 12 observations on 20-in. gauge lengths, however, the maximum variation was only from —3.3 to +1.0%, a range of 4.3 per cent.

When it is remembered that these observations are already averages of four, six, or eight gauge-length readings, it seems fair to inquire, if in as few as thirteen built-up specimens, a 10% range in the average value of E for the whole column is observed, and a 15% range on individual gauge lengths, whether it is very extravagant to suppose that a possible 20% difference in E might occur between the two flanges of a built-up column in practice. Presuming that the ends of such a member were fixed in direction, and the writer's suggestion (that one-half the shift of the center of resistance be regarded as initial curvature, and one-half as eccentricity) is adopted, the eccentricity would not affect the strength, and the proposal would be equivalent to taking the 10% range found by the Committee entirely as initial curvature. Even so, if such a member were short, the effect of the difference in modulus would be the larger part of the initial curvature. In this connection it may be recalled that Fidler took the variation in the modulus as the sole imperfection.

It would be of much interest if the Committee would give the actual difference in E between the two flanges of these thirteen columns, averaging the value over the whole length of each flange.

The question immediately arises as to whether it is necessary to assume that the member suffers from all possible imperfections at once, and, if so, whether it is necessary to assume that each imperfection reaches the maximum value. Professor R. H. Smith‡ suggested an application of the theory of probabilities, and proposed to take seven-elevenths of the maximum total eccentricity, acting in the worst possible direction.

The question is really dependent upon the factor of safety. In a tie or a beam, one factor of safety will cover both accidental increases in the load and imperfections in the material. In a column, accidental increases in

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 415.

† *Loc. cit.*, p. 417.

‡ "The Strength of Struts," *Proceedings*, Edinburgh and Leith Eng. Soc., 1878, Vol. IV, p. 42.

the imperfections, or under-estimates of their magnitude, also have to be covered, and, in addition, the non-proportional increase in the deflection which accompanies accidental increases in the load. The Committee's suggestion* that the same factor of safety should be used for columns as for the other parts of the structure, basing the strength on the yield-point stress, really implies that the true conditions as to eccentricity and crookedness are known; and unless a special factor of safety is to be applied to average values of these imperfections (as some have proposed), to cover possible exceptional material or exceptionally bad straightening, a liberal estimate of their magnitude should be made—such, indeed, as is not in any circumstances likely to be exceeded in practice. It might be urged further that, since in the ordinary short structural column the stress due to bending is small, an over-estimate is not a serious consideration.

Much the same arguments appear to apply in connection with one of the most significant results of the Committee's research, in which it confirms the large variation in the yield point which occurs in ordinary structural mild steel. In discussing the later Watertown Arsenal experiments, Howard repeatedly calls attention to similar variations, and their effect in bringing about local failure in built-up members.

While the Committee may fairly be said to have established that, for practical purposes the tensile yield point may be taken as the failure stress, either a low minimum value should be taken as the yield stress, or if the average value is taken, a reduction of 15 to 20% should be allowed to cover the possible variations. In other words, the Committee's results lend support to the rather usual practice of adopting a lower working stress in compression than in tension.

Assuming then that a suitable value had been taken for e, and that the column has position-fixed ends, the Committee's use of the secant formula to calculate the stress at yield in the column—as indeed is proved by the experiments—is perfectly sound. The method is probably sufficiently accurate in the case of a direction-fixed column provided that a suitable value is taken for the "free length". In seeking an approximation to replace the secant, the writer believes that it is justifiable to take into consideration the fact that, due to the relative shortness of ordinary structural columns, only a small part of the secant curve has any relevance, and that part is very flat. A simple formula can then be used to determine the stress in the column. Fig. 41 has been plotted to illustrate this point. The abscissas represent the ratios of $\frac{p}{p_e}$, and the ordinates, $\frac{f}{p_e}$. Thus, the abscissas represent to some scale the load per square inch on the column, and the ordinates the maximum stress in the extreme fibers. If the column remained absolutely straight up to Euler's crippling load, p would be equal to f, and the maximum stress would be represented by the straight line, OE, at 45 degrees. If the column bends, the stress will increase on the concave side

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 424.

and decrease on the convex. This is shown in the heavy curved lines, which are plotted from the formula:*

$$\frac{f}{p_e} = \frac{p}{p_e}\left[1 \pm \frac{c}{r^2}\left\{e \sec \frac{\pi}{2}\sqrt{\frac{p}{p_e}} + \frac{8\,p_e\,e'}{\pi^2\,p}\left(\sec \frac{\pi}{2}\sqrt{\frac{p}{p_e}} - 1\right)\right\}\right] \quad ..(32)$$

The first half of Equation (32) will be recognized as the secant formula for eccentric loading, the second half is the corresponding expression for initial curvature. Four cases are shown corresponding to $\frac{e\,c}{r^2} = \frac{e'\,c}{r^2} = 0.05, 0.1,$

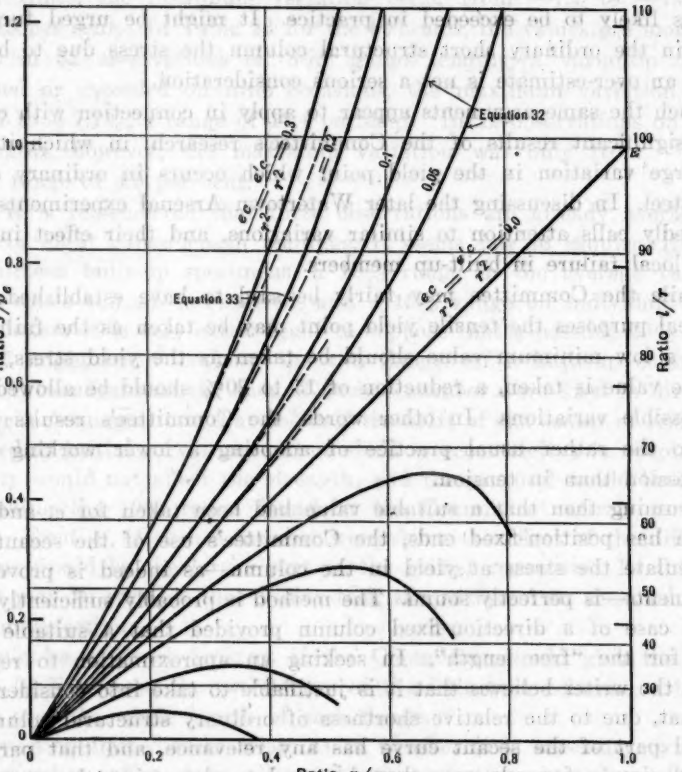

FIG. 41.

0.2 and 0.3, respectively. It is assumed for this purpose that, of the total deviation from axial loading, one-half is due to eccentricity and one-half to initial curvature. The last case is approximately equivalent to $\frac{e\,c}{r^2} = 0.6$, representing, according to Moncrieff,† the worst experimental columns, and

* "Columns," Equation (86), p. 42.

† "The Practical Column Under Central or Eccentric Loads," *Transactions*, Am. Soc. C. E., Vol. XLV (1901), p. 334.

taken by him as a suitable value for a practical column. On the same diagram is plotted the formula:*

$$\frac{f}{p_e} = \frac{p}{p_e}\left[1 + \left(1 + \frac{3\,p}{2\,p_e}\right)\frac{c}{r^2}\,(e + e')\right] \quad \ldots\ldots\ldots\ldots (33)$$

which is represented by the light broken lines. It will be seen that up to the value, $\frac{p}{p_e} = 0.3$, the agreement is excellent; up to $\frac{p}{p_e} = 0.4$, the agreement is good; and even at $\frac{p}{p_e} = 0.5$, for the greatest value of $\frac{e\,c}{r^2} = \frac{e'\,c}{r^2}$, the error is only 10.3 per cent. The ratio, $\frac{p}{p_e}$, is the factor of safety in Euler's formula, that is, against instability, and for ordinary structural columns this is not likely to be less than 5, or $\frac{p}{p_e} = 0.2$; for the shorter columns, it will be much greater. A series of horizontal lines has been drawn on Fig. 41 representing the value of $\frac{l}{r}$, for which, $f = 30\,000$ lb. per sq. in., the minimum value for the yield point of the steel used in the Committee's experiments, plotted from the formula:

$$\frac{f}{p_e} = \frac{f}{\pi^2\,E}\left(\frac{l}{r}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

It will be seen that for the worst case, $\frac{e\,c}{r^2} = \frac{e'\,c}{r^2} = 0.3$, when $f = 30\,000$ lb. per sq. in.; that is, at the failure point, if $\frac{l}{r} = 100$, the value, $\frac{p}{p_e}$, is only equal to 0.46, and if $\frac{l}{r} = 50$, the value, $\frac{p}{p_e}$, is only equal to 0.15.

These values of $\frac{p}{p_e}$ are sufficiently small to justify the use of Equation (33) practically up to the failure point of such columns; and at working stresses the corresponding values of $\frac{p}{p_e}$ will be less than one-half.

Equation (33) is very much simpler than the secant formula, or any of the algebraic substitutes which have been proposed, ingenious as they are, and in view of the fact that a possible variation in f of 33% must be considered and that, at best, e and e' must be probable assumptions, it seems to the writer that the much simpler equation is all-sufficient for practical work provided always the ratio, $\frac{p}{p_e}$, never exceeds 0.5. If it is preferred to treat the deviation entirely as an eccentricity, e, and arrange so that the stress shall be entirely covered by the approximate formula when $\frac{e\,c}{r^2} = 0.6$ and $\frac{p}{p_e} = 0.5$, the

* "Columns," Equation (93 A), p. 44.

constant, $\frac{3}{2}$, may be changed to suit the circumstances, and Equation (33) modified to,

$$\frac{f}{p_e} = \frac{p}{p_e}\left[1 + \left(1 + \frac{5}{2}\frac{p}{p_e}\right)\frac{e\,c}{r^2}\right]$$

but for the majority of practical columns Equation (33) can be used with safety.

For columns direction-fixed at their ends (and most practical columns approximate this condition), the curves representing the increase of stress with increasing values of $\frac{p}{p_e}$ are even flatter (see Fig. 42), and a similar device can be applied with even greater justification.

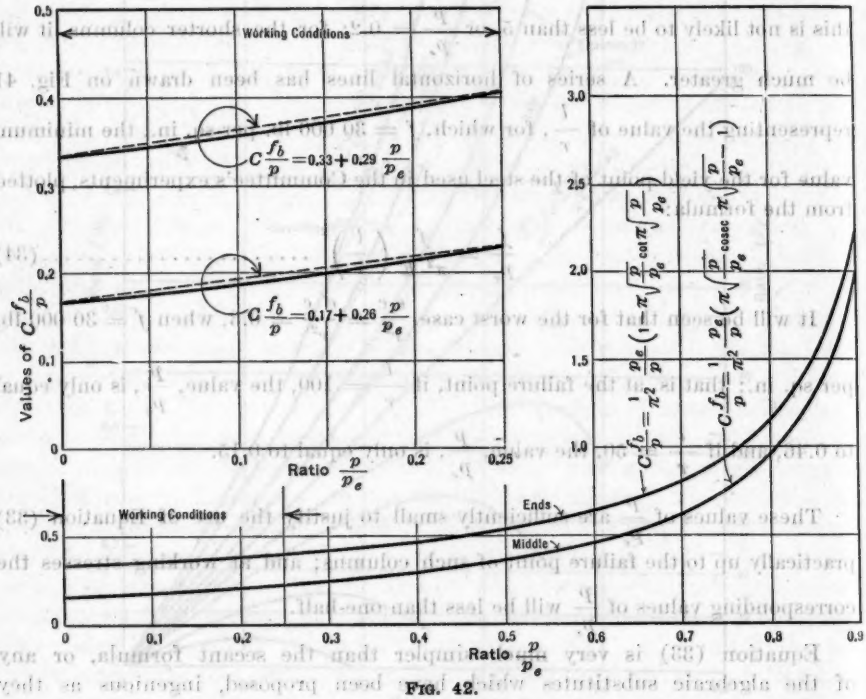

FIG. 42.

Fig. 42 is plotted differently; the base line represents the ratio, $\frac{p}{p_e}$, as before except that, in this case, p_e is $\frac{4\pi^2 E}{\left(\frac{l}{r}\right)^2}$; but the ordinates represent, to a particular scale, the ratio, $\frac{f_b}{p_e}$, in which f_b = stress due to bending. The curves show that for values of $\frac{p}{p_e}$ from 0 to 0.5 at least, the bending stress in a fixed-ended column increases slowly relative to the increasing load. Incidentally, the writer's

diagrams illustrate the fact that for small values of $\frac{p}{p_e}$, that is, under working conditions, the stress due to bending at the ends is double that at the middle, approaching equality when $p = p_e$.*

If use is made of the suggested algebraic equivalents for the unwieldy trigonometrical expressions, there seems no real reason why the fixed-ended column should not be designed rationally instead of by pivoted-end formulas (which at best are rough approximations), particularly if adjustment were made for possible imperfection in the direction-fixing.

Turning to the Committee's observations on shear for axially loaded columns,† the writer considers that this is one of the points in which the "free-length" method of treating fixed-ended columns is apt to be misleading. In a fixed-ended column with parabolic initial curvature, if the origin is taken at its midpoint, the equation‡ for the shear force is:

$$V = \frac{d\,M}{d\,x} = \frac{8\,P\,e'}{\alpha^2\,l^2}\left(\alpha\,k\,\frac{\alpha\,l}{2}\,\text{cosec}\,\frac{\alpha\,l}{2}\,\sin\,\alpha\,x\right) \ldots\ldots\ldots\ldots (35)$$

in which, $\alpha = \frac{2\,\pi}{l}\sqrt{\frac{p}{p_e}}$; $p_e = \frac{4\,\pi^2\,E}{\left(\frac{l}{r}\right)^2}$; and for a column with perfectly direction-fixed ends, $k = 1$. This reduces to:

$$V = \frac{4\,P\,e'}{l}\left[\text{cosec}\,\pi\sqrt{\frac{p}{p_e}}\,\sin\,\alpha\,x\right] \ldots\ldots\ldots\ldots\ldots (36)$$

so that,

$$\frac{d\,V}{d\,x} = \frac{4\,P\,e'}{l}\left[\alpha\,\text{cosec}\,\pi\sqrt{\frac{p}{p_e}}\,\cos\,\alpha\,x\right]$$

and V is a maximum when $\cos\,\alpha\,x = 0$, or $\alpha\,x = \frac{\pi}{2}$, and $x = \frac{l}{4}\sqrt{\frac{p_e}{p}}$. Hence, if $\frac{p}{p_e} = \frac{1}{4}$, $x = \frac{l}{2}$, and the maximum shear, $V = \frac{4\,P\,e'}{l}$, occurs at the ends of the column. This applies also to all values of $\frac{p}{p_e} < \frac{1}{4}$, because x cannot be greater than $\frac{l}{2}$. If $\frac{p}{p_e} > \frac{1}{4}$, there will be a point of inflection in the length of the columns, and the maximum shear will occur there. Its value is expressed by Equation (36) if $\sin\,\alpha\,x$ is made equal to 1.

If, then, the ratio, $\frac{V}{P}$, is plotted on a $\frac{p}{p_e}$ base line, its value will be constant and equal to $4\,\frac{e'}{l}$ up to $\frac{p}{p_e} = \frac{1}{4}$, and then it will increase according to the law, $\frac{4\,e'}{l}\left[\text{cosec}\,\pi\sqrt{\frac{p}{p_e}}\right]$. This is shown by the curve in Fig. 43. For the purpose of plotting this curve, the value of e' has been taken as $\frac{l}{375}$, that is, $\frac{l}{750}$ for initial

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 422.
† *Loc. cit.*, p. 423.
‡ "Columns," Equation (446), p. 193.

crookedness plus an equal amount for variation in the modulus of elasticity. In addition, the values of $\frac{V'}{P}$ given for the American Railway Engineering Association experiments on columns with square ends* have been plotted on the same diagram. For this purpose the specimens have been considered direc-

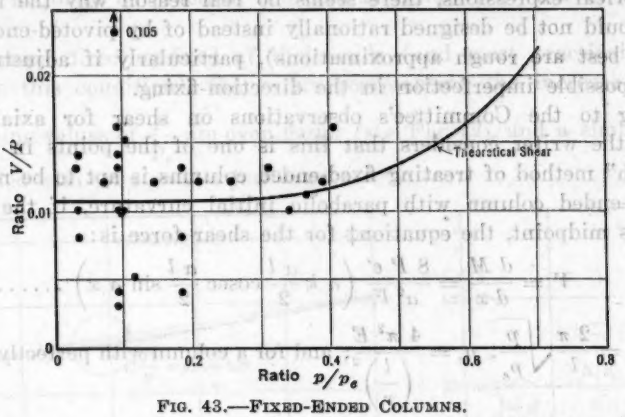

FIG. 43.—FIXED-ENDED COLUMNS.

tion-fixed at their ends. Although the points lie grouped about the curve, too much importance should not be attached to the fact, because the value of e' will be different for each, but with the data at his disposal the writer is unable to make a better estimate. It is difficult to believe, however, that in a practical column the ratio of $\frac{V}{P}$ can increase as $\frac{l}{r}$ diminishes, as suggested by the curve in Fig. 39.†

The theoretical shear in a column intended to be axially loaded is $V = P\frac{dy}{dx}$, and certainly $\frac{dy}{dx}$ will be greater in long columns than in short ones. The Committee attributes the larger values of V, as found from the strain-gauge measurements on the shorter columns, to inequalities in the bearings. Granting this to be the case, the writer would regard it rather as an experimental error not likely to occur with the riveted connections adopted in practice; but he would urge that the American Railway Engineering Association experiments lend no support to the suggestion of increased V for small values of $\frac{l}{r}$ (see Fig. 43). This suggestion comes from the two groups of experiments at $\frac{l}{r} = 22$ and $\frac{l}{r} = 38$ (Fig. 39). In the latter group, made at the University of Illinois, the specimens were abnormal, and had pin ends; in the former, made at the University of Wisconsin, the ends were supported on roller bearings. May not the extra degree of end freedom resulting from pin and roller ends have led to an increase in $\frac{dy}{dx}$, and thus explain, at least in part, the higher

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 448.
† *Loc. cit.*, p. 449.

values of $\frac{V}{P}$ shown by these experiments? One would like further evidence on this point.

Apart from these questions the writer is rather doubtful as to whether strain-gauge measurements on lattice-bars really give an accurate indication of the shear force in columns. The results obtained are quite variable, and are undoubtedly influenced by the flexure of these thin bars. Nevertheless, they are of service in that they confirm the evidence that the shear is small, and that for axial loading the usual assumption of from 2to 2½% of the longitudinal load is approximately correct.

The writer is in thorough agreement with the Committee's proposals relative to future work. He conceives that the proposed investigations under both Part (1) and Part (2)* will lead to just the kind of information that is lacking, and, in particular, is most pleased that it is proposed to investigate the action of actual columns in trusses. Such observations, he is confident, will lead to the establishment of a really rational basis for the design of columns.

J. H. GRIFFITH,† M. AM. SOC. C. E. (by letter).‡—This report presents the data of a carefully conducted series of tests which clarify many points not fully understood in steel column behavior. Among the more important subjects considered, may be mentioned: The effects of the end conditions and eccentric placements of columns in the testing machine; the weights of influence to be attached to irregular deportments of lattice-bars, such as the pantograph action; the proper strengths of lattice and batten-plates; cross-sectional variations; and unintentional cambering of columns. Supplementing its tests, the Committee gives an extended review of important column tests made in the past. This aids much in crystallizing thought as to what constitutes the highest efficiency in column design and fabrication.

Among the various devices discussed, the frictionless end-bearing will commend itself to testing experts. It calls to mind the earlier forms of bearings used by Bauschinger, von Tetmajer, and Considére,§ with which the more modern bearing permits favorable comparison. The new bearing seems very substantial and quite adequate for the heavier column loadings of present-day practice in testing. Although no provision is made for the possibility of some torsion in the columns, which is taken into account in the higher analysis, actual bridge columns in service are largely constrained to displacements in the plane of the truss without twist. A review of the test data shows that these bearings fulfilled the intents of the experimenters and were sensitive enough to respond to very small variations in the intentional eccentricities used in the tests. The determinations of the frictions at the column ends to an order of magnitude of a single percentage seem to be amply sufficient. These findings help to remove one of the chief uncertainties present in column investigations—the indetermination existing at the terminals.

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 459.

† Prof. of Experimental Eng., Iowa State Coll., Ames, Iowa.

‡ Received by the Secretary, April 24, 1929.

§ Discussed by the late J. B. Johnson, M. Am. Soc. C. E., in the earlier editions of his book, "The Materials of Construction".

The apparatus used for the oblique loading tests is unique and quite simple in construction. Tests of the kind submitted should be appreciated by architectural and mill engineers who, of necessity, must load small columns with extreme eccentricities, as for example, the columns used to support the light crane runways of machine shops. The specimens tested in oblique loadings showed the irregular and discontinuous kind of failures expected in the case of batten-plate and lattice-bar constructions in some instances; in others, they showed the familiar reverse curves for elastic lines as described in various treatises.

Some discrepancy arises between the theoretical derivations and the actual results experienced when latticed columns are tested transversely. This shows one type of inefficiency, among the many others, which occurs in lattice-bar constructions. The methods of using bubble levels commends itself in measuring the slopes of the elastic lines for columns in service that are tested in conformity with Manderla's analysis for secondary stresses. This method has been used in Germany to some extent and should be accorded a wider usage because it gives a direct measurement of the variation in curvature which is the cause of the strain usually determined by extensometers or strain gauges.

The apparatus used for obtaining the stresses in lattice-bars, which are treated as small columns individually, appears to secure results that simulate in behavior (although in a magnified form) the type of strain which will actually take place in the column. It is easily constructed and applied to the ordinary testing-machine platens. Until engineers design a more efficient form of transverse bracing than the lattice-bar type with its many anomalies and enigmas of deportment, its seems desirable to make as many investigations as possible, so that the material supplied for the purpose may be ample. If once a device is used, however unsatisfactory, it tends to persist by a kind of inertia and is very difficult to displace however conscious most engineers may be of the numerous defects, such as those in lattice construction.

The analyses submitted and the remarks pertaining to the integral behaviors of columns exposed to strain are usually of most interest to engineers. The question naturally arises, in the light of what has been reported by the Committee, as to what is the most approved form of column equation? Furthermore, does the formula actually check with the test results? Is the formula completely determinate and what details best realize integral behavior? The Committee is apparently inclined to the opinion that the well-known secant formula is the most useful and determinate when the necessary data are obtainable. This was shown quite clearly in the H-sections, the agreement between calculations and tests being all that could be expected, considering the numerous variables that are really under consideration.

A fair confirmation between theory and tests was found in the case of some large bridge columns of alloy steel reported by the U. S. Bureau of Standards.* When the results were interpreted according to methods sug-

* *Technologic Paper No. 101*, U. S. Bureau of Standards.

gested by Henry S. Prichard, M. Am. Soc. C. E., and Dr. O. H. Basquin, some very interesting agreements were found relative to predicted and actual results. The large columns tested followed the secant law until the pin-plates slipped or until the end plane of the column ceased to rotate about its peripheral edge of contact with the upper testing machine platen so that the joint closed. Then, because the eccentricity became practically constant, the stress was relatively constant. This phenomenon was occasioned by the difficulty of getting a uniform contact with the platen and would undoubtedly have disappeared if a roller bearing of the type used by the Committee had then been available. The reason that the stress-strain diagrams in such cases are slightly curved instead of being straight lines is because the assumed mathematical rotation about the outer edge is, in fact, a rocking which is probably caused by the line contact becoming blunted under the column pressure resulting in a slight changing of the eccentricity taking place under stress.

The secant column formula is, in the writer's opinion, the logical consummation of the Bernouilli-Euler or elastic line analysis as applied to compression members. When the equation is developed in the series form by expanding the secant expression, it shows, by a term-to-term comparison (neglecting appropriate terms of higher order), the degree of rationality that attaches to the various short formulas that have been suggested, namely, the short-column formula, the Johnson "straight-line" formula, the parabolic formula, and, as a limiting case, the well-known Euler expression for very slender columns. While the Rankine formula cannot be thus included it is an easy matter to show the type of discrepancy that occurs. The writer believes that if the secant formula were developed in elliptical or Weierstrass sigma functions as was once suggested by Dr. George R. Olshausen in correspondence at the time of tests of the large bridge columns previously mentioned, such an extension would represent the Bernouilli-Euler elastic line analysis to perfection. It would include all the other shorter formulas based on the elastic line analysis to the extent in which these short formulas are admissible as logical derivations according to the law of expressing the bending moment as a function of the curvature. It would serve as the norm of comparison for elastic line column analyses as fast as careful analyses were presented.

In its summary the Committee re-affirms the opinion, advanced from time to time in earlier papers, that the tensile elastic limits of test specimens give the criteria for determining the compression strengths for columns of moderate slenderness ratios. This rule was followed with satisfactory results by most of the consulting engineers, who observed the Bureau of Standards tests, in their estimates of the strengths. Not merely the elastic limits and yield stresses, but the maximum and ultimate stresses, are really conditioned by the rates of loading; therefore, the time factor should enter column analyses as a parameter. In making calculations of the stresses and strains in test pieces, the speed of the straining heads on the testing machine (from 0.05 in. to 12 in. per min.) should be taken into account. The strength of a column under test is likewise affected by the speed of the straining head. The apparent strength may be augmentd by increasing the speed.

Items 19* and 20† of the Committee's "Summary of Results" call attention to the difficulties of securing integral action in the column and an appropriate distribution of stresses as the result of the presence of lattice-bars. As the bar systems shorten under pressure caused by pantograph action, they force the column ribs to deflect laterally, which is a behavior not contemplated in efficient design. To be sure the intermediate bars introduced by the late C. H. Cartlidge, M. Am. Soc. C. E., on the test columns for the Metropolis Bridge,‡ eliminated this entirely and did much to insure integral action. Nevertheless, there are compressions introduced in the lattice-bars, as mentioned by the Committee, together with unsatisfactory secondary stresses and alternating actions under stress, that cannot very well be avoided. Since the writer believes that lattice-bars appear to have more disadvantages than advantages,§ it is pertinent to ask why they should be retained in column construction. Mr. James E. Howard once showed‖ that, under normal conditions they serve no function in taking stress.

It would seem to the writer that lattice-bars and their attendant evils may be easily omitted altogether: (1) By the use of transverse diaphragms or cross-braces spaced at intervals equal to about twice the depth of the member; and (2) by a suitable reinforcement of the rib flanges in the plane of the truss, not only to maintain a properly balanced section about the column centroid but also to carry any shear normal to this plane. On the side of the column where the lattice-bars are to be replaced there should be a transverse strut of the type used by Mr. Cartlidge‡ between lattice panels in maintaining the integral behavior of the column. Such transverse diaphragms and struts would effectively stiffen the column and reinforce it against the possibilities of twisting. It would insure unity of action in the manner evidently intended by the late Mr. Cartlidge, thoroughly holding the open face flanges in line. The construction would be freer of access for inspection and painting and the diaphragms could be made ample to carry any normal shear, such as that due to heavy wind pressure. There would be none of the anomalous behaviors that are now encountered in the case of lattice-bars. Extra longitudinal ribs or diaphragms could be added if desirable. While the diaphragms would introduce a greater efficiency of behavior, the writer believes the cost of construction would be less.

The two suggestions under the paragraph "Suggestions Relative to Future Work"§ are very pertinent. It is very important to insure a continuity of action within the steel as well as in the components and details. Since, on the large alloy-steel columns previously mentioned, all the pin plates slipped and there was evidence of considerable mechanical hysteresis, it might be well to consider the possibility of welding the pin plates and other component parts in addition to the riveting of columns. In the best class of work, the present riveting and reaming practice should be continued to insure continuity and unity of behavior. The costs and labor saved by the omission of lattice-

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 458.
† *Loc. cit.*, p. 459.
‡ *Technologic Paper No. 101*, U. S. Bureau of Standards.
§ *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 459.
‖ *Transactions*, Am. Soc. C. E., Vol. LXXIII (1911), p. 492.

bars, etc., could be applied to the extra expense incurred by electro-welding. Moreover, it is a maxim in the science of dynamics, first enunciated by Kelvin* and later re-affirmed by Lamb,* that the irrotational motion of a liquid, within a region having a single boundary, is accompanied by the storing of less energy consistent with the same normal motion of the boundary. George F. Swain, Past-President, Am. Soc. C. E., and Mr. Frank Cilley long ago stated† the closely analogous proposition that structures with the fewest statically indeterminate component parts are not only the most efficient carriers of stress, but that they are the most economical to construct. This fact is recognized in most conservative specifications.

The writer believes that the time has come when the elastic line analysis for bridge columns may be very favorably superseded by so-called "field methods". When structures are composed of members that are "trunk-line" carriers of stress, without a multiplicity of small statically indeterminate members, it is easy to determine the types of fields by means of the method much used and well illustrated by Bach.‡ He divided a number of his specimen faces into fields of small squares or rectangles which, by their distortions under strain, showed the types of strain occurring in the pieces. In like manner the faces of the column ribs and diaphragms may be similarly divided into small squares or rectangles. In fact, this was done to some extent on the alloy-steel columns tested by the U. S. Bureau of Standards, but the relative inelastic displacements under stress largely rendered the efforts inoperative, causing large sets and hysteresis. Love§ proves the theorem that, when the surface displacements of the surface tractions are measurable, the problem is uniquely determinate.

The proposed method is one which readily adapts itself to the strain-gauge measurements used extensively by A. H. Fuller, H. C. Berry, George E. Beggs, Members, Am. Soc. C. E., and Professor H. L. Whittemore. The displacement functions, the components of which (measured in the direction of the coordinate axes, say, x, y, and z), are usually designated as u, v, and w, may be found by summing the respective elongations of the small squares, for example, of sides 8 or 10 in. long. The strains, expressed as the partial derivatives, $\frac{d\,u}{d\,x}$, $\frac{d\,v}{d\,y}$, and $\frac{d\,w}{d\,z}$, etc., are found by differentiating the surfaces: $u = u\ (x, y, z)$; $v = v\ (x, y, z)$; and $w = w\ (x, y, z)$. Having found the strains or displacements, it is a simple matter of differentiation to obtain the stresses expressed as functions of the position variables, x, y, and z, referred to a suitable origin on the column faces. Numerous examples of procedure are given by Love§ and by Prescott‖ in a manner more suitable for the use of engineers. Attention is called to interesting plats of the surfaces obtained by similar methods herein proposed on a celluloid model of the Stevenson

* "Hydrodynamics," Third Edition, p. 45, and footnotes.

† *Massachusetts Institute Technology Quarterly*, Vol. 10, 1897, No. 2, p. 267.

‡ *"Elasticität und Festigheit."*

§ "Theory of Elasticity," Second Edition, p. 167.

‖ "Applied Elasticity."

Creek Dam by Professor Beggs* and by W. A. Slater, M. Am. Soc. C. E.† and to the type of analysis to be used by H. M. Westergaard, M. Am. Soc. C. E., in his analyses of the same arch dam.‡

The column analyses may be simplified if the stress equations are suggested by the secant type of formula, or by Manderla's extension of the elastic line theory as applied to trusses, the latter affording the cue to the class of functions desired. The writer has applied these methods with excellent results to complicated problems in earth resistance, in which large strains and hysteresis phenomena are present. The problems of the column are simplified by eliminating strain hysteresis in the manner already discussed. Moreover, one need only consider the first-order strains of the theory of elasticity in deriving the surfaces.§ The addition of many small members will, of course, render such methods practically prohibitive by complication of the processes.

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 224.

† *Loc. cit.*, pp. 133, 135, *et al.*

‡ *Loc. cit.*, p. 221 *et seq.*

§ See treatment of simple surfaces by writer in "Proportioning of Pit-Run Aggregates for Concrete," *Engineering News-Record*, March 10, 1927, p. 399; also, in "Pressures under Substructures," *Engineering and Contracting*, March, 1929.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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FINAL REPORT OF THE SPECIAL COMMITTEE ON IMPACT IN HIGHWAY BRIDGES

Discussion*

By Llewellyn N. Edwards, M. Am. Soc. C. E.

Llewellyn N. Edwards,† M. Am. Soc. C. E. (by letter).‡—This report contains instructive and valuable information. However, it seems unfortunate that the research tests and the data secured therefrom, which have been studied by the Committee, did not cover a wider range of highway bridge types, span lengths, and conditions of loading. A large proportion of American highway bridges are short-span structures, or are composed of multiple spans of short length. Incidentally, a considerable percentage of the superstructures of these bridges are constructed of reinforced concrete and are of either the slab or the T-beam type. In general, reinforced concrete slabs are used for spans varying from 2 ft. to 20 ft. in length and T-beams from 21 ft. to 45 ft., with occasional structures ranging up to 60 ft., in length. The reinforced concrete through-girder type of superstructure is used to a limited extent. The test data studied by the Committee contain little information that can be construed to apply to these structures.

It is mentioned in the report§ that, apart from the physical irregularities involved in roadway floor surfaces, the design of the structure itself was found to be a factor involved in the determination of impact stresses. From examinations of the physical condition of many bridge structures, the writer has been forced to recognize that not only adequacy in the proportioning of the sections of the main members of bridges, but also an equal liberality in the proportioning of the detail parts forming the connections between the individual members, is of paramount importance in the service functioning of these structures. Until more is known definitely about the ways in which

* Discussion of the Final Report of the Special Committee on Impact in Highway Bridges, continued from May, 1929, *Proceedings*.

† Senior Highway Bridge Engr., U. S. Bureau of Public Roads, Fort Worth, Tex.

‡ Received by the Secretary, April 22, 1929.

§ *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 740.

different types of bridge structures and their details of construction function under loadings, it seems hardly proper to predicate impact allowances upon other than a basis that applies to all types of highway bridges. Designers cannot assume with safety that the vehicles which pass over any structure will be equipped with wheels of a kind likely to produce minimum impact effects, nor can they assume that the materials or the methods of construction used will produce structures of an assumed good practice standard, or better. Irregularities in the surfaces of bridge roadways may result from poor workmanship incidental to their original construction, but there are perhaps more instances wherein failures of the surface resulting from traffic wear and tear have produced real obstructions to vehicle movement equal in magnitude and in many cases greater than the 1 in. by 2-in. obstruction used in the bridge impact test. However, apart from these conditions, it is fairly common, in areas where small stones and pebbles abound, to find these obstructions scattered promiscuously upon the surface of the roadway.

For motor trucks and other vehicles provided with springs, the impact effort is evidently produced by the unsprung load and the intensity of the spring pressure; but for other wheeled loads, as, for example, heavy road rollers, shovels, and other heavy portable equipment, there is no sprung load factor involved. In general, the speed rate of movement of these loadings of the latter class is comparatively slow, but frequently the loads are moved by truck traction rather than by their own power and in such cases speed may become a factor.

The Committee recommends* "that for the design of highway bridge floors and floor-beam suspenders, the impact increment of stress be assumed as 25% of the live load stress", and suggests that this percentage should be used only when the floors are sufficiently smooth to conform to good modern practice. The designer can have no way of knowing just what will be the character of the roadway surface produced on the bridge for which he, perhaps, does not even prepare the construction detail plans.

The curves shown in Fig. 3† of the report are perhaps misleading in relation to the Conference Committee (A. R. E. A. and A. A. S. H. O., 1927) impact formula since this formula applies to all span lengths, instead of only to those having effective lengths of 50 ft. and more. In speculations and meditations upon this problem of impact the writer has held rather rigidly to the idea that, in bridge structures, length of span—as a factor tending to provide a means of diffusing, distributing, and absorbing its dynamic effects—is an element to be considered. With this idea in mind he has assumed, naturally enough, the fundamental condition to be that of the impact effect of traffic on a pavement as applying practically without diminution to bridges of short span—in the culvert class—wherein the superstructure is commonly either built monolithically with the substructure, or is very rigidly supported thereon. This seems to be a logical assumption and is quite evidently the basis of the formula promulgated by the Joint Committee.

* *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 732.
† *Loc. cit.*, p. 734.

Incidentally, the initial 40% impact increment chosen by the Committee seems to be justified by the tests.

In the investigations* of the U. S. Bureau of Public Roads, the tests made with the object of co-ordinating the results secured by the use of the impact machine with results obtained by using motor trucks developed a condition which may be regarded as pertinent to the question under discussion, since they show a relation between rate of speed and height of obstruction, or its equivalent in irregularities of roadway surface. These comparative tests show that, in general, the variations in impact reactions resulting from speeds varying from 4 to 12 miles per hour are practically equal to the variations resulting from obstructions ranging in height from 0.33 in. to 1.38 in. ($\frac{3}{8}$ in. to $1\frac{3}{8}$ in.).

The impact test results studied by the Committee indicate rather sharply how much engineers should know concerning bridge structures, which has not yet been discovered by actual research tests and a study of the information so obtained.

* *Public Roads*, Vol. 9, No. 6, August, 1928.

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USE OF WATER ON FEDERAL IRRIGATION PROJECTS

Discussion*

By Messrs. Robert A. Monroe and Ivan E. Houk.

Robert A. Monroe,† M. Am. Soc. C. E. (by letter).‡—This paper should be very useful to engineers engaged in irrigation work since it presents reliable long-term records showing the duty of water for a large area and under wide variation of climatic and soil conditions. Since all these twenty-four Reclamation Service Projects are under one central authority, it is possible to have systematic and complete records kept, and the data presented can be accepted unquestionably as accurate and dependable.

Only two of these projects are located in California: The Klamath Project which lies on the boundary between Oregon and California, and the Orland Project at the upper end of the Sacramento Valley. A great many valuable data along the same lines as that presented by Mr. Debler have been collected by the State of California and are available in published form.§

The Company with which the writer is connected supplies irrigation water to a large area of land in Placer County, California, and while complete statistical data, such as are presented by Mr. Debler, are not available, it is thought that the following information will be of value in indicating the duty of water in this section.

The area irrigated lies at an elevation of from 300 to 1 500 ft. and is in the rolling foothills in a fan-shaped section of land lying between the Amer-

* This discussion (of the paper by E. B. Debler, M. Am. Soc. C. E., published in March, 1929, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Chf., Div. of Civ. Eng., Pacific Gas & Electric Co., San Francisco, Calif.

‡ Received by the Secretary, May 7, 1929.

§ *Bulletin No. 6* issued by the Department of Public Works, Division of Engineering and Irrigation, entitled "Irrigation Requirements of California Lands", being Appendix B to a Report to the Legislature of 1923 on the Water Resources of California, contains much information pertaining to the duty of water, and *Bulletin No. 8*, entitled "Cost of Water to Irrigators in California", by Harry F. Blaney, Assoc. M. Am. Soc. C. E., issued by the Department of Public Works, Division of Engineering and Irrigation in 1925, presents data showing the cost of irrigation for nearly every irrigation system of the State.

ican and Bear Rivers. The crops raised consist almost entirely of deciduous fruits; it has been estimated that 99% of the net area irrigated is covered by orchards. The soil can be classified as largely granitic in origin.

The water supply system was originally developed for hydraulic mining operations which were formerly very extensive in this region. After the mines were closed the water was naturally diverted to irrigation use, and with the advent of hydro-electric development between 1890 and 1900 several small plants were constructed to utilize the flow through such head as could be developed cheaply en route. In recent years both storage and conduit capacity have been greatly increased; there is now a sustained water supply of approximately 350 sec-ft., which is first used for power through a chain of plants under a total head of 3 700 ft., and finally becomes available for irrigation use at the tail-race of the lowest plant.

The storage is located high in the Sierras, where there is a total of more than 150 000 acre-ft. impounded in about 22 reservoirs. There are also 23 regulating reservoirs on the system. The canal system consists of 326 miles of conduit, of which 58 miles may be considered as the main conduit carrying water for both power and irrigation, and the remaining 268 miles are irrigation laterals. The topography is broken, and the irrigation laterals include many long stretches of pressure pipe.

The gross area that lies under the ditch system approximates 85 000 acres. The area actually irrigated has shown a rather steady annual increase from 13 500 acres in 1914 to 27 100 acres in 1928. Water deliveries for irrigation purposes are made to the consumers at a uniform rate during five months of the year beginning May 1 and ending September 30. No record is available as to the actual quantity of water used each month, as the consumer contracts for a certain number of inches for the season and rotates it on his land as he sees fit. Frequently, several owners who take water from a common lateral will combine to rotate the water on their holdings.

The duty of water is well established and the contracted delivery is almost generally estimated on the basis of 1 miner's in. of water (0.025 sec-ft.) to 5 acres of land, which for uniform flow is 0.3 acre-ft. per acre per month, or 1.5 acre-ft. per acre for the season. The price which the Company receives for this water is $45 per miner's in. per season of 5 months, which makes the cost of water $9 per acre per year, or $6 per acre-ft. No charge is made for water furnished during the non-irrigation season and most of the canals are kept in service the entire year to deliver the small quantity of water necessary for domestic purposes.

The conduit loss was formerly about 45%, but this has been reduced in recent years due to extensive concrete lining and plastering of sections of canals and replacement of wooden flumes with metal; for 1928 this loss was estimated at 37 per cent.

The water supply to date has been ample to supply the area irrigated, no curtailment having been necessary even in years of extreme drought such as occurred in 1924, and in most years a substantial quantity is by-passed into the American River. This volume of spill is gradually decreasing as the irrigated area increases, but it seems that the water now available will sub-

stantially supply all the irrigable area which is immediately tributary to the system.

IVAN E. HOUK,* M. AM. SOC. C. E. (by letter).†—Comprehensive data on the use of irrigation water are needed by all engineers engaged on the design or investigation of irrigation systems. Therefore, the detailed information presented so concisely in this paper, will be a valuable aid to all practicing irrigation engineers in the Western United States.

The data regarding irrigated areas, miles of canals operated, soil conditions, monthly deliveries, character of crops, canal losses, etc., included in Tables 1 to 25,‡ have a pertinent value in showing the conditions under which irrigation is conducted on the various widely scattered projects. However, the most valuable data included in the tables are the total depths of water delivered to the farms plus the depths of rainfall during the growing season. Reference to Table 25 shows that with the exception of the Umatilla Project, of Oregon, the Uncompahgre Project, of Colorado, and the King Hill Project, of Idaho, the average quantities delivered, plus the growing season precipitation, vary from a minimum of 1.56 ft. on the Milk River Project, of Montana, to 4.09 ft. on the Grand Valley Project, of Colorado. The corresponding quantities for the Umatilla, Uncompahgre, and King Hill Projects are 5.37, 6.31, and 7.36 ft., respectively. The latter values are relatively high because of the sand and gravelly soils and excessive waste of irrigation water.

The total depths of precipitation during the growing season apparently are not very great as compared with those of irrigation water delivered to the farms plus the depths of growing season precipitation. These quantities may be referred to as "total depths of water applied to the farms", or simply "total depths applied". The average values of growing season precipitation recorded in Table 25 are usually less than one-third the total depths applied, the only two exceptions being the Belle Fourche Project, of South Dakota, where the growing season precipitation amounts to approximately 41% of the total depth applied, and the Milk River Project, of Montana, where it amounts to about 58% of the total depth applied. The actual depths of rainfall during the growing season vary from an average of 0.16 ft. on the Tieton Division of the Yakima Project, Washington, to an average of 0.91 ft. on the Milk River Project, Montana.

The total depths of water applied to the farms would be expected to vary, in a general way, with the length of the growing season, although, of course, no mathematically definite relation could be anticipated because of variations in soil conditions, character of crops, elevation of project lands, mean annual temperature, and other meteorological conditions, all of which are interrelated in a more or less complex manner. In Fig. 2 the total depths of water applied are plotted as abscissas against the lengths of the growing seasons as ordinates, the lengths of the growing seasons being taken the same as in Tables 1 to 25. Although the plotted points are somewhat scattered, as would be expected,

* Research Engr., U. S. Bureau of Reclamation, Denver, Colo.
† Received by the Secretary, May 31, 1929.
‡ *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, pp. 753–778.

they indicate a general increase in total depth applied with an increase in length of growing season, especially in the case of projects where the length of the growing season does not exceed eight months. In the case of the Rio Grande and Carlsbad Projects, of New Mexico, where the length of the growing season is ten months, and in that of the Yuma Project, of Arizona and California, where it is twelve months, the increased length of growing season does not require an increased application of water. Apparently, there are two reasons for this: First, a reduction in the quantities of water delivered to the farms during the winter months, when the meteorological conditions are relatively less favorable for plant growth than during the other months of the year; and, second, a reduction in the quantities delivered during some of the summer months, due to a period of dormancy in plant growth caused by the existence of unusually high temperatures. Of course, the points representing the total depths applied on the Umatilla, Uncompahgre, and King Hill Projects, where waste of water and return flow are excessive, fall at the right of the diagram, some distance from the points representing the total depths applied on the other projects.

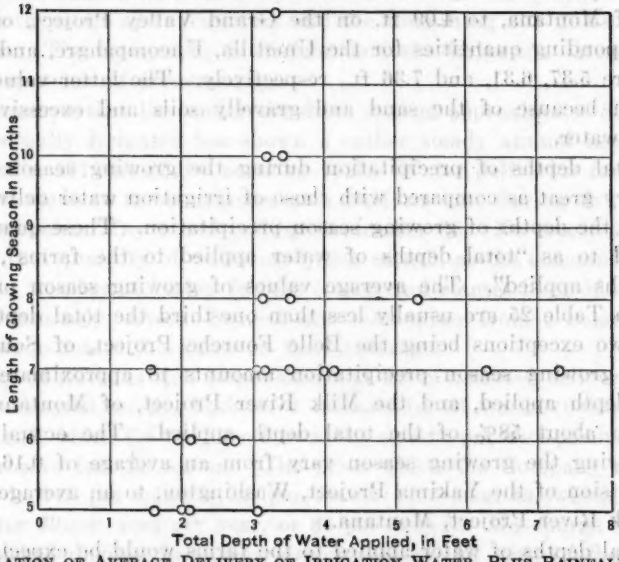

FIG. 2.—RELATION OF AVERAGE DELIVERY OF IRRIGATION WATER, PLUS RAINFALL, TO LENGTH OF GROWING SEASON.

A question that naturally arises in considering Fig. 2 is as to what proportions of the quantities applied constitute consumptive use. The same conditions that affect the depths of irrigation water delivered to the farms affect the depths of consumptive use, although not necessarily in the same degree. For instance, the mechanical texture of the soil which is probably the most important factor as regards quantity of water delivered, assuming an unlimited supply, has relatively little affect on consumptive use, other things being equal. If accurate information regarding depths of consumptive use were available for the different projects, and if the values were plotted on Fig. 2,

the points would probably fall somewhere at the left of those representing total depths applied, the exact position depending on the definition adopted for consumptive use. If consumptive use is defined as the sum of plant transpiration and soil evaporation during the growing season, the consumptive use points would necessarily fall at the left of those representing total depths applied, since some return flow undoubtedly occurs on all projects. However, if consumptive use is defined as the sum of plant transpiration and soil evaporation during the entire year, as would appear to be more reasonable, it is conceivable that some of the consumptive use points might fall at the right of those representing total depths applied, since the total depths of rainfall during the non-growing season, which would be added to the total depth applied during the growing season, in determining the consumptive use, might be greater than the total depth of return flow, which would be subtracted.

In either case the writer would expect the consumptive use points to be more consistently grouped than the points representing total depths applied, assuming, of course, that proper corrections are made for variations in soil storage. Adding the non-growing season precipitation to the total depths applied, as given in Table 25, and leaving out of consideration the three projects on which total deliveries are excessive, gives total yearly depths applied ranging from an average of 1.74 ft. on the Milk River Project, Montana, to an average of 4.57 ft. on the Orland Project, California. These values would be the sums of the consumptive use and the return flow, assuming the variations in ground-water storage to be rendered negligible through the averaging process.

In regard to the depths of consumptive use required for the satisfactory growth of crops, the writer believes that much useful information can be obtained by a study of rainfall and run-off records in Central and Eastern United States.* There are appreciable differences between the Central and Eastern sections of the country, and the Western irrigation projects, as regards some of the meteorological conditions which affect plant growth, particularly relative humidity and percentage of cloudiness, not to mention rainfall which, of course, is more or less equalized by the delivery of irrigation water. However, it is not believed that the total depths of water required to raise the same crop are materially different in the two sections of the country. Experiments on the water requirements of plants by Montgomery and Kiesselbach, Heinrich, and Sorauer, referred to by Messrs. Lyman J. Briggs and H. L. Shantz, showed that the quantity of water used per pound of dry matter produced was greater in the dryer atmosphere.† However, they also showed that the number of pounds of dry matter produced was greater in the humid atmospheres, so that the total quantities of water used were not so greatly different. Cloudiness seems to reduce the rate of transpiration, but to increase the total water requirement per pound of dry matter produced, although there is some uncertainty in this respect due to the varying rates of photo-synthesis.

* "Evaporation on United States Reclamation Projects," by Ivan E. Houk, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 371.

† "The Water Requirements of Plants," by Lyman J. Briggs and H. L. Shantz, *Bulletin 285*, Pt. II, Bureau of Plant Industry, U. S. Dept. of Agriculture, 1913.

TABLE 26.—CONSUMPTIVE USE OF WATER IN MIAMI VALLEY, SOUTHWESTERN OHIO.

Gauging station.	Stream.	Drainage area, in square miles.	Mean yearly rainfall, in feet.	Mean yearly run-off, in feet.	TOTAL EVAPORATION,* IN FEET, DURING 12-MONTH PERIODS.†					Average evaporation or consumptive use.
					1915.	1916.	1917.	1918.	1919.	
Sidney	Miami River	555	3.02	0.98	2.35	1.57	2.18	2.17	1.98	2.05
Lockington	Loramie Creek	255	2.98	1.11	....	1.43	1.77	2.08	....	1.88
Piqua	Miami River	842	3.03	1.05	2.22	1.57	2.07	....	1.91	1.98
Tadmor	Miami River	1 128	3.08	1.10	2.35	1.62	1.95	2.23	1.79	1.98
Pleasant Hill	Stillwater River	453	3.15	1.11	....	....	2.06	2.09	1.74	2.04
West Milton	Stillwater River	600	3.13	1.13	2.59	1.67	2.07	1.84	1.84	1.99
Buck Creek	Buck Creek	163	3.22	0.90	2.65	2.28	2.00	2.39	2.22	2.32
Springfield	Mad River	488	3.31	1.07	2.48	2.20	2.05	2.34	2.14	2.24
Wright	Mad River	652	3.29	1.15	2.40	1.95	1.91	2.34	2.10	2.14
Dayton	Miami River	2 525	3.15	1.05	2.48	1.72	2.03	2.28	1.95	2.10
Franklin	Miami River	2 785	3.15	1.05	....	....	1.99	2.33	1.95	2.10
Germantown	Twin Creek	272	3.19	1.22	2.15	1.67	1.98	2.28	1.80	1.97
Seven Mile	Seven-Mile Creek	128	3.29	1.62	2.01	1.30	1.83	1.79	1.42	1.68
Four Mile	Four-Mile Creek	178	3.27	1.27	2.06	1.23	2.02	2.58	2.06	1.99
Hamilton	Miami River	3 672	3.17	1.04	2.54	1.75	2.07	2.36	1.92	2.13

* Including interception, evaporation from soil and water surfaces, and plant transpiration.
† Ending September 30.

Table 26 gives some data on depths of annual consumptive use in the Miami Valley of Southwestern Ohio, for the five years from 1915 to 1919, inclusive, compiled from one of the Technical Reports of the Miami Conservancy District.* The consumptive use values were computed from carefully determined isohyetal maps and elaborate stream flow measurements, and, consequently, are believed to be more accurate than most consumptive use data thus far published. The drainage areas represented in the table are more than 90% agricultural land, only comparatively small areas being occupied by cities, towns, farm buildings, roads, rivers, lakes, reservoirs, etc. The length of the growing season, which is approximately the same in all parts of the valley, averages about six months, and paying crops are produced each year. Crops consist primarily of corn, wheat, rye, oats, barley, alfalfa, and tobacco. The acreages of the three classes of crops considered in the author's Tables 1 to 25 are approximately equal and are, therefore, about the same as on the North Platte Project, of Wyoming and Nebraska. The topography of the Miami Valley may be described as gently rolling with the ground elevations of the uplands varying from about 800 ft. above mean sea level near the mouth of the river to about 1 100 ft. near the head-waters. The slopes are comparatively flat near the head-waters but they increase more or less gradually toward the southwest, being comparatively abrupt near the Ohio River. The mean annual temperature of the valley is approximately 53° Fahr., the mean annual relative humidity about 67%, the mean annual rainfall about 37 in., the mean annual run-off about one-third the rainfall,

* "Rainfall and Run-off in the Miami Valley," by Ivan E. Houk, M. Am. Soc. C. E., Miami Conservancy Dist. Technical Repts., Pt. VIII, pp. 134 to 150, Dayton, Ohio, 1921.

and the mean annual evaporation, or consumptive use, about two-thirds the rainfall.

Rainfall and run-off records were compiled for years ending September 30, that being the time of the year when the quantity of water stored in the ground is least variable. Average yearly totals for the different drainage areas, based on the five years from 1915 to 1919, are included in Table 26. Differences between the yearly rainfall and run-off totals, heretofore usually termed "evaporation", but which are really consumptive use values, are given in Table 26 for each of the five years mentioned. The drainage areas in the table are arranged according to location, beginning at the upper end of the valley. The drainage area of 3 672 sq. miles above Hamilton, the last station in the table and the one farthest down stream, for which rainfall and run-off data were compiled, includes all the other drainage areas listed in Table 26. Consequently, the values of average rainfall, run-off, and consumptive use for Hamilton are really averages for all the other stations in the table. For this reason no final line of average data for all stations has been included. However, a final column of average consumptive use for the five years of record has been added.

The average values of consumptive use are seen to vary from a minimum of 1.68 ft. on the Seven-Mile Creek drainage area to a maximum of 2.32 ft. on the Buck Creek drainage area, averaging 2.13 ft. for the entire valley above Hamilton. Values for single years vary through slightly greater ranges, but at least a part of the yearly variations are undoubtedly due to variations in ground-water storage, a factor which is believed to be negligible as far as the 5-year averages are concerned. The consumptive use values include plant transpiration, soil evaporation, interception losses, and evaporation from river, lake, and reservoir surfaces, all but the first two being of minor importance. Since the data were compiled for the yearly periods, they include evaporation from soil, snow, and ice surfaces during the winter months, quantities which amount to about 4 in. on the average, or to about 15.5% of the average annual consumptive use in the valley above Hamilton. R. E. Horton, M. Am. Soc. C. E., who has made elaborate studies of interception, estimated that the seasonal interception losses on an average Michigan farm were equivalent to an average depth of 1 in. of rainfall over the entire farm, which would correspond to about 4% of the average consumptive use above Hamilton.* It is interesting to note that the 5-year average consumptive use of 2.13 ft. for the drainage area above Hamilton, is not greatly different from the 9-year average of 2.06 ft. for the same area; and, also, that the 5-year average of 2.10 ft. for the drainage area above Dayton, Ohio, is exactly the same as the 25-year average for the same area.† These comparisons of the 5-year averages with the averages based on longer periods supports the claim that the data in Table 26 have been accurately determined.

* "Hydrology of the Great Lakes," by R. E. Horton, M. Am. Soc. C. E., in collaboration with C. E. Grunsky, Past-President. Am. Soc. C. E., Rept. of Engineering Board of Review of the Sanitary District of Chicago, 1927, Pt. III, Appendix II, p. 46.

† *Loc. cit.*, pp. 140 and 142.

MEMOIRS OF DECEASED MEMBERS

FAYETTE SAMUEL CURTIS, Past-President, Am. Soc. C. E.*

DIED FEBRUARY 15, 1929.

Fayette Samuel Curtis sprang from early American stock. Henry Curtis, his paternal ancestor, came from England in 1635, settling first at Watertown and then at Sudbury, Mass.; his mother's family also were early settlers of that State.

Fayette Samuel Curtis, the son of Allen and Catherine (Steel) Curtis, was born on December 16, 1843, at Owego, N. Y., and lived there the first twenty years of his life, attending the district school until he was twelve, then private school for two more years, and, finally, Owego Academy, where he received special instruction in Civil Engineering.

Finishing with Owego Academy in 1863, Mr. Curtis was employed on the location and construction of a line for the Albany and Susquehanna Railroad Company between Binghamton and Albany, N. Y., first as Rodman, and then, in 1864, as Transitman. Two years later, he went with the Southern Central Railroad Company on its line from Owego to Auburn, N. Y., serving as Levelman during the location, and then, with the commencement of construction, as Assistant to the Resident Engineer. On the completion of this work he accepted a position as Transitman with the Lake Ontario Shore Railroad Company on location between Oswego and Charlotte, in New York State.

In the spring of 1869, as Assistant to the late George S. Greene, Jr., M. Am. Soc. C. E., young Curtis was employed on surveys in the Towns of Morrisania, Tremont, and Fordham, now parts of Greater New York, which surveys have since served as the base of much of the later mapping in The Bronx. After two years of this work, in April, 1871, he returned to the railroads, as Assistant Engineer, under the late I. C. Buckhout, M. Am. Soc. C. E., on the New York and Harlem Railroad, and six months later, on October 1, he became Principal Assistant, first on the location of the tracks and yards at the old Grand Central Station, in New York, N. Y., and, later, on the plans and the construction of the Fourth Avenue Improvement between the Grand Central Station and the Harlem River.

On the retirement of Mr. Buckhout in 1874, Mr. Curtis became Chief Engineer of the Harlem Railroad Company and Superintending Engineer of the Fourth Avenue Improvement, continuing in the service of the former Company until the completion of its line. The Harlem Railroad, about this time, was leased to the New York Central and Hudson River Railroad Company, and while Mr. Curtis was still acting as Engineer for the Harlem Lines, the grain elevators, yards, docks, piers, sheds, and warehouses on the Hudson River, between Fifty-ninth and Seventy-second Streets, were designed and built for the New York Central under his direction.

On December 1, 1882, Mr. Curtis was called to the New York, New Haven, and Hartford Railroad Company, as Chief Engineer, with headquarters

* Memoir prepared by Clarence Blakeslee, M. Am. Soc. C. E., assisted by E. J. Beugler, C. C. Elwell, Edward Gagel, H. C. Keith, H. L. Ripley, Henry B. Seaman, Paul Sterling, and E. W. Wiggin, Members, Am. Soc. C. E.

first at New York, and later at New Haven, Conn. This position he held until May 1, 1900, when he became Fourth Vice-President in charge of "engineering and such other matters" as might be assigned to him by the President or the Board of Directors, his headquarters being at Boston, Mass.

As of June 1, 1903, his duties were more specifically set out as follows: "General supervision of the Engineering Department and construction, including the Electrical Department, except operation, also maintenance of way, and construction and maintenance of buildings and signals, except the operation of signals". This assignment remained unchanged when, on November 14, 1903, Mr. Curtis was made Second Vice-President, with headquarters at Boston, and also when, a year later, he was transferred to New Haven. As of October 1, 1905, however, he returned to Boston, to "represent the President with full authority in all matters requiring his attention", and remained in this position for two years. On April 1, 1907, he resigned the Vice-Presidency of the New York, New Haven, and Hartford Railroad Company to accept the Presidency of the Old Colony Railroad Company, which latter position, together with those of President of the Union Freight Railroad Company, of Boston, and Chairman of the Boston Terminal Company, he held at the time of his death.

Mr. Curtis came to the New York, New Haven, and Hartford Railroad Company very largely through the influence of Mr. Charles P. Clark, then Second Vice-President, who already had well in mind plans for a unified transportation system for Southern New England, and who correctly appraised the worth of Mr. Curtis as an assistant in the development. The latter had but just come to the property, however, when Mr. Clark accepted a position with the New England Railroad Company and the next four years in the engineering affairs of the New York, New Haven, and Hartford Railroad Company were marked merely by consistent rather than by extensive improvements.

In the spring of 1887, however, Mr. Clark returned to the New York, New Haven, and Hartford Railroad Company, as President, and immediately began to build up and to expand the property. In this work Mr. Curtis played a most important part as one of the group of trusted lieutenants who served not only in the fields specifically assigned to them, but as general advisers and assistants as well. How highly his judgment was esteemed is shown by an endorsement on a letter in the office files of the Company. President Clark had referred an inquiry to Mr. William E. Barnett, then Executive Secretary and Attorney, who returned it with the note: "I do not know; ask F. S.; he has second sight if any one has."

In December, 1882, when Mr. Curtis came to the New York, New Haven, and Hartford Railroad Company, the line had double track from New York to Springfield, Mass., approximately 123 miles, and single track elsewhere, the total single-track equivalent being but a little more than 400 miles. This was laid with 60-lb., or lighter, rail, most of the latter being iron. The great majority of its highway crossings were at grade, and in the year ending September 30, 1882, it had operated in round figures 185 000 000 passenger miles and had moved 117 000 000 ton-miles of freight.

By 1907, the line from New York to New Haven had been completely four-tracked, as had that part of the Providence Division from Readville into

Boston, so that at the close of 1907, when Mr. Curtis' assignment to Boston as "representative of the President, with full authority in all matters requiring his attention", made it desirable to relieve him of close control of engineering, there were being operated about 4 254 single-track equivalent miles with 89⅔ miles of line four-tracked and 651¾ more, double-tracked, largely laid with 100-lb. rail on stone ballast. The section of this rail was designed by Mr. Curtis in 1893, its height of 6 in., with a 6-in. base, giving extra stiffness and stability. There was not a single grade crossing on the four-track sections, from New York to New Haven, and that in the vicinity of Boston, and a large number elsewhere on the System had been eliminated, notably in connection with very extensive improvements at Westerly and Providence, R. I.; Hartford, Conn.; and Attleboro, Brockton, Fall River, and Worcester, Mass.; while the traffic had so increased that in 1907 there were operated roundly, 1 256 000 000 passenger miles, and 1 889 000 000 ton-miles of freight were moved.

This, however, is but the merest outline of the story of railway improvements for which Mr. Curtis largely was responsible. With much of the work in thickly settled communities, and all of it adjacent to, if not directly under, heavy traffic, engineering ability of the highest quality was demanded, both of the Chief and of his assistants. How well the test was met by Mr. Curtis' professional skill and the keen discernment with which he picked his lieutenants, is clearly shown in the fact that in the conduct of this great work not a single passenger was killed or seriously injured; and this record was practically duplicated as regarded the general public, except as to accidents resulting from the personal carelessness and disregard of barriers and warnings by the injured persons.

Nor is it without significance that, except as in some instances prospective increased loadings far above those for which they were designed have necessitated the replacement of some of the earlier structures, there have been neither failures nor indications of weakness in any of the structural work. This required that every contractor be held rigorously to his agreed performance, but it is quite characteristic of the fairness with which Mr. Curtis maintained his standards that in not a single instance was there an appeal to higher authority, either within the Company or without, relative to his decisions.

Carrying to an extreme his dislike for any publicity, there is on record very little regarding the engineering features of this work thus broadly outlined. To the engineer familiar with New England conditions, however, the bridges, particularly over the tidal estuaries and the larger rivers, the elevated work through Bridgeport, Conn., Attleboro, Brockton, and the approach to Boston, the "cuts" at Mount Vernon, N. Y., New Haven, and in Boston; the stations at Bridgeport, Hartford, and Providence, as well as many less important—all tell unmistakably of the hand of the master engineer.

Mr. Curtis also played an important part in the design of the South Station, Boston. He, with L. B. Bidwell, Chief Engineer of the New York and New England Lines, and Walter Shepard, Chief Engineer of the Boston and Albany Railroad Company, representing the three roads for which the station was built, constituted the Consulting Engineering Board for that project. In 1895, Mr. Curtis accompanied President Clark and a party of

officers of the New York, New Haven, and Hartford Railroad Company to the Fifth International Railway Congress in London, England. At the close of the Congress he, Mr. Bidwell, and Mr. Shepard, visited the more important railroad stations of England and the Continent, studying their operation, their advantages, and their shortcomings, later employing the knowledge thus secured to the great advantage of Boston.

"F. S." as he was very generally called, was a man of tremendous energy and splendid constitution. Up to less than a year before his death the average man of 65 would have found it very difficult to keep up with him. This was the more remarkable because he had never spared himself. His "day" ended when the work permitted, without reference to the time, and during the lifetime of Mr. Clark a long active day in the field or office frequently was followed by conference until early morning with his equally tireless President.

Possessed of a kindly disposition and a keen sense of humor, Mr. Curtis had the happy faculty of making and keeping friends. His consideration of, and advice to, the young engineers who came under his direction were such as to inspire them to high ideals of engineering accomplishment, and there are many one-time associates who owe their later successes to his early training. With all his duties and responsibilities he always found a minute in which to greet even casual acquaintances, and to exchange the latest stories, his hearty and contagious laugh telling his keen enjoyment of a good tale. He was very fond of horses, and was a regular patron of the old Morris Park Race Track, taking unusual pleasure in attending the races with a crowd of his particular cronies.

He was a member of the Connecticut Society of Civil Engineers, which he had joined on January, 9, 1900, and of which he was made an Honorary Member on February 19, 1924.

Socially, he was a member of the Beacon Society, the Boston Athletic Association, and the Engineers Club of New York; his membership in the Quinnipiack Club of New Haven, which he had long held, he resigned about a year before his death.

Fraternally, Mr. Curtis was a prominent Mason, holding membership in Friendship Lodge, Harlem, in Jerusalem Chapter, of Tioga County, and in Constantine Commandery, Knights Templars, all in New York State.

On October 19, 1872, he was married to Mary J. Bishop, of New Haven, who survives him, as does a sister, Kate S. Curtis, of Owego, and a half-brother, Robert Curtis, of New York. Of his three children, Wallace, George Henry, and Mary J., only the latter, who is the wife of Capt. John D. Woodworth, Medical Corps, U. S. A., at Vancouver, Wash., is now living. There are, however, five grandchildren: John Fayette, son of Wallace; George Henry and Dorothy M., children of George; and Fayette Curtis and Mary Justin, children of Mrs. Woodworth. Mr. Curtis' death, after an illness of three months, occurred on February 16, 1929, at his home in Jamaica Plain, Boston.

An engineer and executive of outstanding knowledge and ability, an indefatigable worker, a staunch and loyal friend, and a gentleman, his death is

a great loss to his friends, to his profession, and to all who had the privilege of being associated with him or knowing him.

Mr. Curtis was elected a Member of the American Society of Civil Engineers on April 3, 1889. He served as Director from 1895 to 1897, inclusive; as Vice-President in 1904 and 1905; and as President in 1919.

WILLIAM STUART AUCHINCLOSS, M. Am. Soc. C. E.*

DIED APRIL 10, 1928.

William Stuart Auchincloss, the son of John and Elizabeth Auchincloss, was born in New York, N. Y., on March 19, 1842. His primary education was obtained at the Bacon School and at Lyons School, both in New York. Because of his decided mechanical turn of mind he then entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1862, at the head of his class, with the degree of Civil Engineer.

In order to secure the advantage of being well grounded in shop work, Mr. Auchincloss began his career as an employee of the Novelty Iron Works, of New York, where large marine engines were built. At the time, the Company was engaged on the construction of a steamer for the Pacific Mail Route and the iron-clad, *Roanoke*, for the United States Navy.

In 1867, he was appointed by the Government as Commissioner to the Paris Exposition. While abroad on this important errand, he visited Great Britain, France, Germany, Austria, Italy, Egypt, and Palestine. He spent several months at the Exposition and made a complete report to the Government on his return to the United States.

For several years Mr. Auchincloss was connected with the Jersey City Locomotive Works, the Atlantic and Great Western Railway Company, and the Ramapo Car and Foundry Company. After conducting the sale of the Jersey City Locomotive Works, he bought an interest in the Delaware Car Works, of Wilmington, Del., and became Vice-President of the Company.

In 1874, Mr. Auchincloss visited Brazil and built the rolling stock of the São Paulo and Rio de Janeiro Railway. In 1876, subsequent to his return to Wilmington, the Emperor, Don Pedro II, of Brazil visited the United States and was entertained at the home of Mr. and Mrs. Auchincloss. While the Emperor was in this country he ordered a private car built by the Delaware Car Works, which was designed and built under the supervision of Mr. Auchincloss. This car was exhibited at the Philadelphia Centennial Exposition in 1876 and received first prize.

Mr. Auchincloss moved from Wilmington to Philadelphia, Pa., in 1879, and organized the firm of Bates and Auchincloss, Agents of Messrs. J. and P. Coats, Thread Manufacturers, of Paisley, Scotland. After Mr. Bates died, Mr. Auchincloss carried on the business in his own name, establishing his family, and building a home, in Bryn Mawr, Pa., which he called "Hillcrest".

* Memoir prepared by William K. Auchincloss, Esq., New York, N. Y.

In 1903, having retired from business, he moved to Atlantic Highlands, N. J., where he built another home, "Rest Haven", on the shore of Sandy Hook Bay. Here, he lived until his death on April 10, 1928.

Mr. Auchincloss' tastes were decidedly mechanical, but, in later years, he devoted much of his time to literary work. He was the author of the following books and booklets: "Report to U. S. Government, as Honorary Commissioner to the Paris Exposition" (1867); "Standard Work on Link and Valve Motions" (1869); "90 Days in the Tropics" (1874); "Schieber und Contissendteurungen" ("Link and Valve Motions", published in Germany, 1886); "Yearly Tides" (1892); "Waters within the Earth and Laws of Rainflow" (1897); "Stoomschief-en Schaarbewegingen" ("Link and Valve Motions", published in Holland, 1899); "St. Peter, the Apostle of Asia" (1901); "The Book of Daniel Unlocked" (1905); "The Mask of Eddyism" (1907); "Standard Chronology of the Holy Bible" (1908); "Wonders of God's Universe" (1912); "Seaside Astronomy" (1920); "Christianity and the Britons" (1920); "The Greatest Man of the Ages" (1921); and the final edition of the "Standard Chronology of the Holy Bible" (1924).

Mr. Auchincloss also directed his genius to the invention of an averaging machine in 1882. This machine had a capacity of 110 business accounts per hour, which equalled the customary task of six skilled accountants. A paper describing the machine, entitled "Averaging Machine",* was presented by its inventor before the Society. Relative to his mechanical work, Mr. Auchincloss had received six patents.

In 1870, he was married to Martha Tuthill Kent, the daughter of William Campfield Kent, a noted merchant of the firm of James Kent Santee and Company, of Philadelphia. They had three children, James Stuart, Jane Kent, and William Kent.

He was most kindly, upright, and honorable in all his dealings—a devoted father, and a Christian gentleman.

Mr. Auchincloss was elected a Member of the American Society of Civil Engineers on February 7, 1869.

JAMES BURDEN, M. Am. Soc. C. E.†

DIED MARCH 1, 1928.

James Burden was born at Glasgow, Scotland on January 12, 1864, the son of John and Janet (Duncanson) Burden. When he was very young his parents came to the United States. He received his preliminary education in the public schools of Troy, N. Y., and at the Sterling, Scotland, High School; he was graduated from the Rensselaer Polytechnic Institute at Troy, in 1892, with the degree of Civil Engineer.

After his graduation from college, Mr. Burden made a specialty of construction work on dams, canals, and water-works. He was engaged for a time as Draftsman for the Metropolitan (Boston) Water-Works in connection with

* *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 121.

† Memoir prepared by Morton Burden, Assoc. M. Am. Soc. C. E.

the Wachusett Aqueduct and Reservoir. In 1897 and 1898, he had charge of the New York State improvements on the Minetto Dam and on the canal at Manlius Center, N. Y. In 1899 he entered the employ of the Sharon Steel Company of Sharon, Pa., for which he designed and installed the sewers, water supply, and railroad yards at its new plant.

From 1900 to 1904, Mr. Burden served as Principal Assistant Engineer on the construction of the Troy Water-Works, in charge of the surveys and construction of the pipe lines and dams. From 1904 to 1908, he was Assistant Engineer in the main office of the New York State Barge Canal, at Albany, N. Y., where he made studies of back-water above proposed dams across the Mohawk River as well as of the design of locks and movable dams. In 1908 Mr. Burden was appointed Resident Engineer on various sections of the New York State Barge Canal, at Mechanicsville, Oswego, and Utica, N. Y., which position he retained until 1918. From 1919 to 1921, he was employed with the Emergency Fleet Corporation at Washington, D. C., and Philadelphia, Pa., in connection with dry docks and marine railways. The following year he was appointed City Engineer of Oswego, which position he held until his death.

Mr. Burden was a member of Grace Presbyterian Church of Oswego, and took an active part in church work, serving as President of the Board of Trustees. He was also a member of the Permanent International Association of Congresses of Navigation; the Fortnightly Club, and the Oswego Country Club. He was sincere in all his undertakings, giving his best to everything with which he was connected.

On December 5, 1912, Mr. Burden was married to Edith Sloan, the daughter of the late James and Mary Sloan, of Oswego, who survived him. Mrs. Burden died on October 23, 1928.

Mr. Burden was elected an Associate Member of the American Society of Civil Engineers on February 5, 1902, and a Member on January 5, 1904.

EDWARD BERTIE CODWISE, M. Am. Soc. C. E.*

DIED FEBRUARY 11, 1927.

Edward Bertie Codwise, the son of Elisha Rogers and Harriette M. S. (Salter) Codwise, was born at Elizabeth, N. J., on May 9, 1849. His ancestors came to the United States from Holland in 1624, and settled in Massachusetts. In 1664 the family moved to Elizabeth. His great-grandfather, Christopher Codwise, was a field officer in the Continental Army and a member of the Society of the Cincinnati.

Mr. Codwise received his education at the Baldwin Academy, in New York, N. Y., from which he was graduated. Later, he attended the Brooklyn Collegiate and Polytechnic Institute and was graduated with the Class of 1865. Subsequently, he was for two years a student at the L'Institution Davaux, at Amiens, France.

On his return to the United States in 1868, Mr. Codwise accepted a position as Assistant Engineer with the Erie Railroad Company on the con-

* Memoir prepared from information on file at the Headquarters of the Society.

struction of the Newburgh and New York Railroad, the Fort Lee Railroad, and the Weehawken, N. J., Cattle Yards. From 1869 to 1870 he served as Assistant Engineer with the New Jersey Southern Railroad Company, and from 1870 to 1872 he was engaged as Principal Assistant Engineer for the Wallkill Valley Railroad Company on construction work from Gardner to Kingston, N. Y.

Mr. Codwise spent a brief period from March, 1872, to April, 1873, in Peru, South America, where he was Assistant Engineer on the Callao, Lima, and Oroya Railroad. He was engaged a part of this time on the design and construction of what proved to be, for many years, the longest and highest railroad bridge in the world.

After his return to this country, he held for a short period the position of Principal Assistant Engineer for the New Jersey Southern Railroad Company. In 1874, he opened a private office in Rosendale, N. Y., and also served as Engineer for the Rosendale Cement Company. In 1876 he gave up this occupation to accept the position of Assistant Engineer in charge of inventory surveys, etc., for the Erie Railroad Company.

From 1879 until his retirement in 1923, Mr. Codwise held the following positions: Assistant Engineer in charge of the construction of the New York City Elevated Railroad on Ninth Avenue between 30th and 59th Streets; Division Engineer for the Albany and Jersey City Railroad (now the West Shore Railroad) Company on location surveys; Assistant Engineer in charge of the construction of extensions of the Wallkill Valley Railroad; Resident Engineer of the West Shore Company in charge of construction from Newburgh to Kingston, N. Y.; Chief Engineer of the Ulster and Delaware Railroad Company; and City Engineer of Kingston, where he was also engaged in private practice. During this time, Mr. Codwise designed the present water supply system of Kingston. His choice for the location of the reservoir was between two sites, one of which contemplated that a dam be placed across Esopus Creek only a few hundred feet down stream from the site of the present Ashokan Dam of the New York Water Supply. The reservoir, however, was constructed at the other location on Sawkill Creek, which was much nearer Kingston and more economical.

Because of a complete collection of original maps of Central and Eastern New York, which date back to the land grants from European rulers, Mr. Codwise was called upon to compile the location of the lands and boundaries of the State Forest Preserve.

He was married on March 28, 1872, to Emma Snyder, of Rosendale, who died in 1924. After his wife's death, he resided with his daughter, Mrs. R. H. Edmondson, of Morgantown, W. Va., who, with his two sons, Henry R. Codwise, M. Am. Soc. C. E., and George Wallace Codwise, also a Civil Engineer, of Kingston, survives him.

Mr. Codwise was exceedingly fond of his family, preferring his home to clubs and social life. He was of a happy, likeable disposition, admired by all his employees and honored by his business associates. His reputation for honesty and integrity was well demonstrated by the confidence that was

placed upon his testimony when he appeared as an expert witness in Court cases. He was a member of the Kingston Club and of Kingston Lodge No. 10, F. and A. M.

Mr. Codwise was elected a Member of the American Society of Civil Engineers on September 5, 1888.

FRANK HILL McGUFFIE, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 17, 1928.

Frank Hill McGuffie, the son of Matthew and Martha (Morgan) McGuffie, was born at Waverly, N. Y., on August 27, 1887. His early life was spent in his native village. He attended St. John's College, Manlius, N. Y., from 1903 to 1906, and for several months in 1913 was a student at Cornell University, Ithaca, N. Y., in special engineering work.

During the summer months, while he was at college, Mr. McGuffie worked as Chainman and Rodman for the Lehigh Valley Railroad Company, and continued in that employ as Rodman and Levelman from 1906 until 1908. In July, 1908, he accepted a position as Engineer in charge of construction on the estate of Mr. B. L. Winters, at Smithboro, N. Y. His work there which continued until October, 1910, consisted of the design and construction of roads, small bridges, and a water supply system. He then spent several months as Draftsman and Transitman on base line surveys for the Riordan Company, of London, Ont., at St. Jovite, Que., Canada.

In May, 1911, Mr. McGuffie entered the service of the New York State Department of Highways and for more than six years was engaged as a Levelman in charge of contracts and survey parties on location. From 1918 to July, 1924, inclusive, he served as Assistant Engineer with the same Department in responsible charge of the design of pavements and bridges. His duties included supervision of village and county paving, highway construction work, and the maintenance of 150 miles of highways. Coincident with this engagement, Mr. McGuffie was County Engineer of Sullivan County, New York, for three years. He also did a certain amount of private work, including the design of a sewerage system for the Village of Waverly, bridge work for the Delaware and Northern Railroad Company, and the design of a dam for the Callicoon Electric Company.

From August, 1924, until his death, he was employed by the National Surety Company of New York as Field Engineer, with headquarters at Denver, Colo. He was sent West by the Surety Company to supervise the completion of a large paving job for the Colorado State Highway Department on which the contractor had defaulted. His work for this company included the making of cost estimates, preparation of plans and specifications, acting in an advisory capacity to contractors, and the compiling of contract data for underwriters. His territory included Colorado and the adjacent States, where he was well and favorably known by both contractors and engineers.

* Memoir prepared by William B. Freeman, M. Am. Soc. C. E.

Mr. McGuffie died at Denver on October 17, 1928, and was buried at Sayre, Pa., which is near his old home. Although troubled with pernicious anæmia to some extent, he was apparently in good health and was active in his duties up to a day or two before his death, which came as a distinct shock to his family and friends.

Because of his genial disposition and pleasing personality he was popular among his friends and business associates. He was generous and, with a natural inclination to be accommodating, was willing at all times to take endless trouble in order to be of service to a friend. Gifted with much ability and energy he gave promise of attaining a considerable degree of success in his profession, had not death terminated his career at a relatively early age.

Immediately on his election to the Society, less than a year before his death, Mr. McGuffie became actively interested in the affairs of the Colorado Section and had demonstrated that he was a valuable addition to the membership. He was also a regular attendant at the meetings of the Cornell Club of Colorado. He was a member of the Masonic Fraternity, a Thirty-second Degree Mason, and a Shriner.

On May 24, 1928, he was married to Reba Burrus, at Denver, Colo. In addition to his widow, he is survived by a sister, Mrs. Lois C. Davenport, of Sayre, and a brother, Dr. Robert N. McGuffie, of Passaic, N. J.

Mr. McGuffie was elected an Associate Member of the American Society of Civil Engineers on December 5, 1927.

FRANCIS HERBERT WRIGHT, Assoc. M. Am. Soc. C. E.*

DIED MAY 29, 1928.

Francis Herbert Wright was born in Memphis, Tenn., on December 8, 1874, the second son of Col. A. D. Wright, formerly a Confederate Officer, and Mary Ellen (Farrell) Wright. Colonel Wright was a lineal descendant of a titled English officer of high rank, who married a daughter of a Spanish Grandee, Gen. Manuel Cardenas, and established his residence on a Spanish grant owned by his distinguished father-in-law—the present site of San Antonio, Tex. Mary Ellen Farrell was the grand-daughter of Capt. John Sappington, an early Mayor of St. Louis, Mo. John Sappington, the third of his line and name in America, served seven years in the American Army during the Revolutionary War. He was a member of the personal bodyguard of General Washington, at Valley Forge, and when the war ended was a Captain on the Staff of General Lafayette. With Daniel Boone and a company of pioneers he was one of the founders of Boonesboro, Ky., and served a term in the Kentucky Legislature before going to St. Louis where he died in 1807, a man of property.

Mr. Wright was educated in private schools, where he was distinguished because of his proficiency in, and liking for, mathematics. He began his engineering career at a very early age on levee surveys and construction along the Mississippi River. In 1891, he was a Rodman and Inspector for the Board

* Memoir prepared by Ernest McCullough, M. Am. Soc. C. E.

of State Engineers, at New Orleans, La., and also for the United States Engineer Office in that city. Then followed work for various railway companies and two years in Oregon, Washington, and Montana, on railway location and construction.

Mr. Wright was employed at the World's Fair, Chicago, Ill., in 1893, and from 1896 to 1899 was an Inspector, Corps of Engineers, U. S. Army, under Capt. (now Maj.-Gen., *Retired*), Mason M. Patrick. While on furlough, in 1898, he was employed for a time as an Assistant Engineer in the office of the City Engineer of Memphis, Tenn. In 1899, he became an Inspector of dredging at Port Royal, S. C., for the Bureau of Yards and Docks, United States Navy, and also engaged on levee construction at Bird Point, Mo., for the Corps of Engineers, U. S. Army. This work ended in 1900. During 1901, he was employed as a Concrete and Masonry Inspector for the Illinois Central Railway Company, entering private practice when that work was completed. In 1902-1903, he served as Assistant City Engineer of Little Rock, Ark., and Assistant Engineer in charge of sewer construction in Helena, Ark. After a short period as Draftsman on location for the St. Louis-San Francisco Railroad Company, he became, in 1903, City Engineer of Helena, Ark., where he remained until late in 1907.

In 1908, Mr. Wright went to Chicago, Ill., where he served as Sales Engineer for about a year for the Northwestern Expanded Metal Company. In 1909 he was employed as Chief Designing Engineer for Mr. Carl Weber, who was engaged in the construction of reinforced concrete chimneys. He left Mr. Weber in 1910 to become Chief Engineer for and, later, a partner in, the Julian S. Nolan Company, designers and builders of two-way reinforced concrete and tile slabs. Mr. Wright patented an L-shaped closer for the open ends of tiles used in the Nolan-Wright System and the Company was very successful until the beginning of the World War in 1914, which caused a slowing down of the building industry. With the entry of the United States into the war, in 1917, the Company suspended operations entirely.

Mr. Wright volunteered immediately for service and was recommended for the rank of Captain in the Engineer Officers Reserve Corps, by the Army and Navy officers under whom he had served as a young man. The physical examination revealed a faulty heart, hitherto unsuspected, and he was not accepted. During the war he worked, in the vicinity of New Orleans, in charge of the construction of wharves and warehouses, as an Assistant Engineer for Ford, Bacon and Davis, later doing considerable appraisal work for the same Company, principally for the valuation of alien property. After the Armistice was signed, he served as a Construction Engineer for Dwight P. Robinson and Company, Incorporated, later leaving that Company to become Engineer and Eastern Representative in the New York District for the Hay-Walker Brick Company. He remained in this position for several years until the firm went into the hands of a receiver. During the period he was associated with the Hay-Walker Brick Company, he invented several types of wall tile, the patents being taken out in the names of other men. He was an Estimator for the Longacre Engineering and Construction Company, Incorporated, for a

time, and then became New York Representative for a patented door, on which work he was engaged at the time of his death.

Mr. Wright was a man of very friendly disposition, one intensely interested in everything that tended toward improvement in the working conditions, remuneration, and treatment of engineers. He had a warm heart and made friends easily. In 1924, he purchased an apartment in a co-operative apartment house in Sunnyside Gardens, Long Island City, N. Y., where he served for a time as an officer of the Co-operative Corporation as well as being active for several terms as an officer of local property owners associations. Nothing was too difficult or disagreeable for him to undertake for the general good of the community, and his death was sincerely regretted by a large circle of friends.

In politics, he was a Democrat and a member of the Anaroc Democratic Club. He had a deeply religious nature and was a faithful and devout member of the Roman Catholic Church.

In 1900, he was married to Mary Alice Smith, in Arkansas. They had two children, Mary Frances, one of the prize winners in the International Turnverein Convention swimming contests in Berlin, Germany, in the summer of 1928, and James Mayo, who is in business in New York, N. Y. Mrs. Wright died shortly after the war and, in 1923, Mr. Wright was married to Sarah Leion, of Norwich, Conn., who survives him.

Mr. Wright was elected an Associate Member of the American Society of Civil Engineers on November 4, 1914.

ROBERT BRUETTE POWELL, Jun. Am. Soc. C. E.*

DIED MARCH 11, 1929.

Robert Bruette Powell, the oldest son of Mr. and Mrs. F. J. Powell, of Wausau, Wis., was born at Fort Atkinson, Wis., on May 23, 1900. He was educated at the University of Wisconsin where he received the degree of Bachelor of Science in Civil Engineering in 1923.

During the summers of Mr. Powell's college work he endeavored to gain as much practical experience in his profession as possible. From June to September, 1920, he was employed as Instrumentman with the Wisconsin Valley Electric Company, and in the summers of 1921 and 1922 he served as Inspector and Surveyor on waterway dredging with the Wisconsin Valley Improvement Company. The following year, from March to May, he was engaged in basin and dam site surveys on the Chippewa River for L. A. DeGuerre, Consulting Engineer, Wisconsin Rapids, Wis. At the termination of this short connection, Mr. Powell accepted a position with the Dixie Construction Company, of the Alabama Power Company, and from May, 1923, to February, 1924, he served as Chief of Party on the location of transmission lines and basin surveys.

In April, 1924, Mr. Powell established a connection with the Electric Bond and Share Company, of New York, N. Y., as Designer in the Hydraulic Divi-

* Memoir prepared by A. R. Wellwood, Res. Engr., Saluda River Hydro-Elec. Development, Columbia, S. C.

sion. While in this employ he received excellent training and experience in various kinds of work, such as preliminary studies, license drawings, investigation of old structures, substructures, power house and switchyard, penstocks, spillway gate and bulkhead sections, surge tank foundations, and economic studies.

After three years with the Electric Bond and Share Company he was employed by Murray and Flood, Engineers, New York City, and sent to Columbia, S. C., for assignment on the Saluda River Hydro-Electric Development in making hydraulic investigations and other preliminary engineering studies. When the actual construction work began, Mr. Powell was promoted to the position of Supervising Structural Engineer and his main assignment was the supervision of the erection of the penstocks and the intakes. These structures were practically completed before his sudden death in an automobile accident, and now stand as monuments to him.

After his death, letters were received from men with whom "Bob" Powell had worked, and from men who had worked under him, which told frankly of the high esteem in which he was held by those who knew him. One man wrote, "I never worked with one more sincere, more exacting, and more faithful to any trust"; and to those who knew Mr. Powell best these were surely his outstanding characteristics.

T. C. Williams, Vice-President of the Lexington Water Power Company, the owners of the Saluda River Hydro-Electric Development, stated:

"He had such a pleasing personality and was so well liked by every one. I was especially impressed with his cheerful willingness to work just as many hours as necessary, and when there was a peak load, he was always ready to go the limit".

Arthur R. Wellwood, Resident Engineer on the Saluda Development for Murray and Flood, and Mr. Powell's immediate superior, said:

"I feel that I have lost a friend as well as a valued co-worker. He had a very bright engineering mind and I considered him an A-1 Engineer. He was due to go a long way in his chosen field. He had been entrusted with very important features of the construction of the Saluda Dam and had done efficiently each task assigned".

William S. Murray, of Murray and Flood, Engineers, wrote:

"Powell had a compelling personality and I have a very difficult time reconciling the passing out of a man who seemed to have laid all the foundations for an interesting and constructive career".

Mr. Powell was elected a Junior of the American Society of Civil Engineers on April 12, 1926.

CARL SPEER, Jr., Jun. Am. Soc. C. E.*

DIED JANUARY 18, 1929.

Carl Speer, Jr., the son of Carl and Anna Speer, was born in Baltimore, Md., on December 21, 1901. His preliminary education was received in the

* Memoir prepared by Abel Wolman, M. Am. Soc. C. E.

public schools of Baltimore. He studied Civil and Sanitary Engineering at the Johns Hopkins University, obtaining the degrees of Bachelor of Science in 1923 and Master of Science in Sanitary Engineering in 1926.

From 1921 to 1922, Mr. Speer was employed as Inspector on construction with the Maryland State Roads Commission. During 1923 and 1924, he served as Junior Engineer with the New York Transit Commission on traffic engineering of subways and elevated lines. From June, 1925, to February, 1926, he was engaged in shellfish investigations for the Maryland State Department of Health. Beginning in July, 1926, and until his death, he was carrying out, as Junior Sanitary Engineer in the Division of Water Safety Control of the City of Chicago, Ill., experimental studies in the treatment of the public water supply.

The writer, as teacher, employer, and friend of Mr. Speer, realizes too well the inadequacy of the foregoing chronological account as a description of an individual of more than usual charm and intelligence. Scientific curiosity and enthusiasm, those rare characteristics of productive men, were possessed by Mr. Speer in an unusual degree. That they would have led him upward to important engineering contributions, his friends and his associates have little reason to doubt. These attributes, coupled with a keen and delightful personality, make his death at the beginning of his career most tragic. The world has lost a promising aide. In all his work, energy and ability were dominant. Ever on the alert for new concepts, he carried on his work with freshness and pleasure. His ideal was to create a value to his fellow man.

On June 26, 1926, he was married to Charlotte A. Stout, of Baltimore. He is survived by his widow, his father, and one sister.

Mr. Speer was elected a Junior of the American Society of Civil Engineers on March 14, 1927.